UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC MODEL STUDIES OF GREEN MOUNTAIN DAM SPILLWAY AND TUBE VALVES

COLORADO-BIG THOMPSON PROJECT COLORADO

Hydraulic Laboratory Report No. Hyd.-229

ENGINEERING AND GEOLOGICAL
CONTROL AND RESEARCH DIVISION

BRANCH OF DESIGN AND CONSTRUCTION
DENVER, COLORADO

FEBRUARY 24, 1947

PREFACE

The hydraulic model studies of the Green Mountain Dam Spillway of the Colorado-Big Thompson project, Colorado, were made in the Denver laboratory. The work was started in July 1938, and completed in December of the same year. The original draft of the report was in memorandum form--Memorandum to J. E. Warnock, from J. H. Douma, dated January 10, 1939.

The calibration of the 44-inch regulating tube valves, 1 to 8.333 scale model, was made in 1943. The spillway discharge curves were prepared in 1944.

Urgent work in the laboratory prevented an earlier completion of the report.

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Denver, Colorado
February 24, 1947

Laboratory Report No. 229
Hydraulic Laboratory
Compiled by: J. H. Douma and
Ben R. Blackwell
Reviewed by: J. N. Bradley

Subject: Hydraulic model studies of Green Mountain Dam Spillway and tube valves--Colorado-Big Thompson Project, Colorado.

INTRODUCTION

The prototype structure. Green Mountain Dam is located on the Blue River, a tributary of the Colorado River, about 15 miles south of Kremmling, Colorado (Figure 1). The dam proper consists of an earthfill structure approximately 1,200 feet in length and 270 feet high at the deepest part of the river. The maximum reservoir capacity is 147,000 acre-feet and is utilized for power and irrigation development.

The spillway, which is designed for a maximum capacity of 25,000 second-feet, is at the left abutment of the dam. The approach to the gate section consists of a curved channel. Three radial gates, each 25 feet wide by 22 feet high, control the flow to the chute. The chute, approximately 1,000 feet long, discharges the flow directly into the river channel below the toe of the dam (Figure 2).

Two 44-inch regulating tube valves, which serve as the outlet works for the dam, are located in the powerhouse.

Scope of tests. Model studies were undertaken with five primary objectives in mind: (1) to establish the minimum radius of curvature of the inlet channel for satisfactory hydraulic performance, (2) to determine the efficiency of the spillway crest and bridge piers, (3) to obtain the most economical and effective hydraulic design of the spillway chute, (4) to determine the extent of downstream erosion and the possibility of employing some type of spillway stilling-basin to prevent scour, and (5) to calibrate the 44-inch regulating tube valves.

Summary and recommendations. The model studies showed that both 200- and 400-foot radius inlet channels were satisfactory. It was therefore recommended that the 200-foot radius be utilized.

Satisfactory performance was obtained in the model from the original design of the spillway crest and bridge piers. Discharge curves were prepared and are presented in this report.

Operation of the original chute design indicated that the chute sidewalls converged too rapidly for satisfactory performance. An improved design was evolved from the model studies.

Three types of stilling-pool structures were studied: (1) without artificial stilling-basin, (2) with roller-bucket dissipator, and (3) with horizontal apron stilling-basin. Model experiments were not made on the horizontal apron type. Both types of artificial stilling-basins tried proved uneconomical for this project. The original design was recommended.

A discharge alinement chart was prepared for one 44-inch regulating tube valve for the outlet works.

THE SPILLWAY

The spillway model. A 1 to 40 scale model of the Green Mountain Dam Spillway was constructed in the Denver hydraulic laboratory. Figure 3 shows the general layout of the model and construction details. A model flood discharge of 2,470 second-feet represented the prototype design flood of 25,000 second-feet.

Approach channel alinement. The spillway approach is curved in plan. Excavated material from the approach channel was used in the earth structure, and since this material was suitably located, the original plan called for a large inlet, provided the excavation did not extend into rock. Should rock be encountered, the plan was to have the smallest radius inlet hydraulically possible.

Operation of 400- and 200-foot radius inlets was studied in the model. Observation and measurements showed the smaller inlet to be as satisfactory, hydraulically, as the larger inlet (Figures 4 and 5). Water surfaces in the two approach-channels appeared the same and profiles at Station 9+90, which are not presented herein, were identical. Spillway discharge coefficients were practically the same for the two inlets. Owing to surface wave disturbances in the 200-foot radius inlet channel, it was considered undesirable to further reduce the radius.

Superelevation of approach channel. Observations and measurements of the water surface in the chute with the level approach floor indicated slightly unsymmetrical flow, which, it was thought, might have originated in the curved inlet. To investigate this point, the velocity distribution ahead of the gate structure was determined by taking velocity measurements at Station 9+90. The velocity distribution was not quite symmetrical, which may have resulted in somewhat uneven chute flow. The discharge, as determined by the velocity-area method, was slightly different for each gate, but the difference was probably insignificant. A study of the unbalanced flow pattern in the chute led to the conclusion that energy per unit width through the right gate was greater than that through the left gate, which was later verified by the measurements.

From the theory of a curved channel, to have equal energy distribution in a transverse section, the bottom must be superelevated towards its minimum radius. Approximate computations by the design section indicated that the superelevation should be about 8 feet. To test the theory, the superelevation was made 4 feet. The result was a decrease in discharge through the right gate and an increase through the left gate. The increase in mean velocity through the right gate was about 25 percent. The flow in the chute became more unbalanced than for the level inlet floor. This is contrary to the theory. It was apparent that other factors also seriously influenced the flow through this type of curved channel. The unsymmetrical change from a trapezoidal to rectangular section is probably significant in the development of the flow pattern.

Although the level inlet floor resulted in a slightly unbalanced flow in the chute, this condition should not be serious as excess freeboard has been provided. The level approach floor was therefore recommended.

Overflow crest calibration. Discharge measurements were first made with the 400-foot radius inlet for all gates open and combinations of one or two gates open. The experimental discharge coefficient for all gates open and 25,000 second-feet was 3.21, which compares favorably with the design coefficient of 3.23. The free discharge curve for this condition is shown on Figure 6. The corresponding maximum reservoir elevation is 7950.12, or 0.12 foot above the design maximum which is within experimental error. For single-gate operation, the center gate showed the largest coefficient, the left gate slightly smaller, and the right gate the smallest. The variation, however, was less than 4 percent.

Discharge measurements were not made for partial gate openings. Data for discharges for partial gate openings were obtained from model studies of Wheeler Dam. (Technical Memorandum Number 407, Hydraulic Model experiments for the design of Wheeler Dam, by J. W. Ball, Figure 29, Page 160.) The discharge curves (Figure 6) are for three gates operated simultaneously. For small gate openings where only one or two gates are open, one-third or two-thirds of the value in the discharge curves may be used with no significant error. At larger openings, with gates at different settings, proportionate values of the rating curve will give reasonable results. Uniform gate operation is recommended at all discharges to maintain symmetrical flow in the chute.

These discharge curves, Figure 6, have been in use in the field. An extensive series of tests is being performed on radial gate discharge measurements in the Laboratory at the present time. More accurate discharge curves are forthcoming.

That the pier design is an efficient one is shown by the fact that the discharge coefficient for the maximum head was increased only 0.7 percent when the piers were removed.

Experimental discharge formulae. Designers select a discharge coefficient to suit the type of overflow crest and plot a rating from figures based on the selected coefficient for all heads. There are several factors which influence the value of the coefficient, of which the head is probably the most important. The experimental variation of the coefficient with head is shown in Figure 7. For very low heads, friction becomes significant in the model, so the model coefficients are smaller than those for the prototype for corresponding heads. It is necessary to study prototype measurements to determine the relationship between model and prototype coefficients for low heads. The percentage difference in experimental and design rating curves, Figure 7, becomes large for small heads, being 80 percent for a prototype head of two feet in this case.

The following equation for the discharge with two piers in place, 2 feet 6 inches in width, was determined to fit the experimental curve:

$$Q = \left[10\left(\frac{H'}{H}\right)^{0.007} - 9\right]\left[1 - \frac{0.0007N}{\left(\frac{H'}{H}\right)^{2}}\right] CLH'^{3/2}$$

For the case without piers:

$$Q = \left[10\left(\frac{H'}{H}\right)^{0.007} - 9\right] CLH'^{3/2}$$

where the nomenclature is the same as shown in Figure 7. The latter formula does not follow the experimental curve closely for small heads, but it might more nearly approach the prototype curve in this region. The validity of these formulae for use in design can only be established through prototype measurements.

Spillway chute alinement. Operation of the originally designed spillway chute showed that the channel convergence downstream from the gate structure was too rapid, resulting in uneven transverse water surfaces over the entire chute length (Figure 8). Water was concentrated in the center, higher than the side walls at Station 12+42, and nearly overflowed the walls at Station 13+53 (Figures 9 and 10A).

Several smaller angles of divergence were tested and the design shown in Figure 11 proved satisfactory. Since the lower portion of the chute was to be in rock, a trapezoidal section with 1/2 to 1 side slopes would be more economical than a rectangular section. Rock was not encountered until Station 15+00 was reached, so the upper converging section was tested with vertical side walls. A transition, from vertical to a 1/2 to 1 side slope, from Station 14+22.19 to Station 15+94.22, represents the minimum length that gave satisfactory flow conditions in the lower part of the chute. The recommended design and resulting water-surface profiles at several stations along the chute are shown in Figure 11. Photographs of the recommended chute design are shown on Figures 10B and 12.

Figure 13 shows the maximum water-surface elevations at either side wall as determined from all gate combinations. The side-wall height was increased 2 feet to provide a minimum experimental freeboard of 5 feet. Entrainment of air will reduce this freeboard considerably.

Water-surface profiles for combinations of one- and two-gate operation are shown in Figure 14. Under no condition did the water come dangerously close to the top of the chute walls.

Comparison of 600-foot radius chute vertical curve with jet trajectories. The theoretical equation for the jet trajectory based on the mean velocity at the beginning of the curve is

$$y = -x \tan \theta - \frac{g}{2V_o^2 \cos^2 \theta} x^2$$

where

$y =$ horizontal distance in feet
$x =$ vertical distance in feet
$\theta =$ angle of the chute floor with the horizontal
$g =$ acceleration due to gravity
$V_o =$ initial velocity in feet per second

This equation was used in designing the vertical curve of the Kittitas Chute, Yakima Project. Recent operation of this chute showed the vertical curve to be entirely too steep. So much water sprang from the chute over the side walls that safe operation was limited to about one-half of the maximum discharge.

To prevent any disruption of the jet, the trajectory should be designed for at least the maximum velocity at the beginning of the trajectory. Assuming a maximum velocity equal to 120 percent of the mean velocity, the trajectory equation becomes:

$$y = -x \tan \theta - \frac{g}{2.88V_o^2 \cos^2 \theta} x^2$$

When a mass of water-air mixture passes over a convex vertical curve at high velocity, the reduction of internal pressure within the mass due to centrifugal action, results in an expansion of the air bubbles within the mass, effectively reducing the action of gravity, which then requires flatter slopes to prevent disruption of the jet. A more conservative design formula would perhaps then be

$$y = -x \tan \theta - \frac{g}{4.4V_o^2 \cos^2 \theta} x^2$$

A comparison of the trajectories based on the above formulas and the design curve, would indicate that there will be very little disruption of the jet in the present design as was the case of the Kittitas Chute, Figure 15. The capacity will not be seriously impaired, as was that of the Kittitas Chute.

Mean water surface, velocity, hydraulic gradient, and value of α along chute. The mean water surface (Figure 16) is drawn from computed depths at the several stations at which measurements were taken. Velocity curves are shown as determined by three methods:

$$V = \sqrt{2gH} \quad ; \quad V = \frac{1.486}{n} R^{2/3} S^{1/2}$$, with n = 0.014; and by experiment.

A model value of n = 0.012 was selected for the painted wood and neat cement surfaces in computing the model energy line. With a prototype value of n equal to 0.014, the prototype loss is greater than the model loss and its energy line falls below the model energy line. Prototype velocities will be slightly less than corresponding scaled model velocities.

A hypothetical experimental energy line may be plotted for the model by adding velocity head to the water-surface elevation. The true model energy line is obtained by subtracting the friction head loss from maximum reservoir elevation. The velocity head coefficient, α , is given by the difference in the hypothetical and true energy lines.

Table 1 shows hydraulic model values, converted to prototype quantities. No attempt has been made to correct prototype values for possible difference in prototype and model values of n because of the uncertainty of these values and the effect of entrained air. The point to be noted is that the velocity head coefficient, α , inherent in the velocity distribution, is important in determining the true energy line for high velocity flow. Values of α are in agreement with those of previous investigations. A point of interest is that α increases in value as the velocity increases, which is in agreement with the theory of increased turbulence for higher velocities.

Table 1

Station	Mean depth D	Hydraulic radius R	Mean velocity V_m	Apparent velocity head $\frac{V_m^2}{2g}$	True* velocity head $\alpha \frac{V_m^2}{2g}$	Velocity head coefficient α
9+90	22.50		13.6			
10+50	10.38	8.24	30.3	[illegible]	15.0	1.06
11+10	8.22	6.69	42.2	27.7	29.2	1.05
11+77	7.14	5.80	56.4	49.4	52.6	1.07
13+00	7.73	5.68	75.3	88.0	96.5	1.10
14+22.19	10.10	6.03	82.5	105.6	123.6	1.17
15+19	10.87	6.33	83.9	109.4	128.7	1.18
15+94.22	11.25	6.40	86.5	116.2	133.0	1.15
16+97	11.10	6.34	88.0	120.2	149.2	1.24
17+96	10.20	6.00	97.3	147.0	181.0	1.23
18+61	9.88	5.86	101.2	159.0	201.0	1.26
19+33	9.76	5.84	102.5	163.2	198.6	1.22

*Difference between water surface and energy line elevations.

Stilling-basin investigations. The original plan was to have the jet shoot out into the river without the use of any type of artificial stilling-basin. Since the canyon walls are of good rock, no excessive scouring was anticipated. There was some concern, however, on the height that gravel bars might form downstream. A bar would raise the tailwater in the powerhouse tailrace, effectively reducing the power head.

A series of runs was made to investigate the nature of the downstream bar (Figures 17 to 21, inclusive). A scouring material, representative of the river overburden, was thought to be approximated by a mixture of 75 percent of 1/4- to 1-inch pit run gravel and 25 percent of Cherry Creek sand. The sand, scaled to prototype dimensions, would be representative of pebbles, and the 1-inch gravel to about 3-foot 4-inch rock.

For no conditions of flood did destructive eddies form at the downstream toe of the earth dam, but scouring occurred well downstream. The resulting gravel bar became larger and moved farther downstream for floods up to 7,500 second-feet. For larger floods, the gravel was carried out of the model. In Figure 17A, a gravel bar-crest envelope is drawn from the experimental data. A tangent line to the envelope becomes horizontal for a flood of about 10,000 second-feet. The significance is that the highest bar formed for this flood. For larger floods, bottom velocities become sufficiently above critical for the bed material so that the bar flattened and will not build up as high.

Due to the complexities inherent in similarity of scour and bed material, the model gives only an indication that a bar of considerable height will be formed. Whether the bar will be as high or higher than the 16 feet shown by the model will depend upon the similarity of scouring and duration and rate of subsidence of the flood. For a slow rate of subsidence, much of the bar may be washed downstream as the downstream tailwater falls.

The best field plan is to investigate the bar after each flood and to determine the feasibility of excavating the bar to regain the lost power head. Figure 17B shows that when the bar, resulting from the first flood, was excavated to the original riverbed, a second flood of the same intensity formed a second bar farther downstream but not as high as the first.

As the jet shoots down the canyon, its excess kinetic energy will be expended in turbulence and boundary friction. When sufficient energy has been expended to result in tranquil flow, a semblance of a hydraulic jump will form. After several small floods and perhaps a large one, the gravel overburden will be carried a quarter of a mile or more downstream and beyond the distance required to produce tranquil flow in the rock channel. With the overburden beyond reach of the high velocity jet, further formation of bars need not be anticipated. Figures 18 and 19 show the natural stilling-pool in operation for four discharges. Figures 20 and 21 show the original model bed and the model erosion for three discharges.

Roller-bucket stilling basin. Because of the possible cost of gravel bar excavation or loss of power head, it was desirable to investigate the feasibility of a roller-bucket stilling-basin. Six bucket designs were tested in the model (Figure 22A, B, and C).

Bucket Design No. 1 had the dimensions: 40-foot radius, 100-foot depth, and 30-foot width. Model operation showed this bucket to be satisfactory for discharges up to 5,000 second-feet.

Bucket No. 6 was the same as No. 1, except that the invert was 20 feet lower (bucket depth 120 feet). This bucket proved satisfactory for 7,500 second-feet. According to the model performance, this bucket should be about 100 feet wide to accommodate a 25,000-second-foot flood.

Buckets Nos. 2, 3, 4, and 5 had the same bucket dimensions as No. 6, but the shape of excavation beyond the end of the bucket was different for each. All of these designs were inferior to No. 6. No. 2 was definitely the poorest design, since the jet followed the 1 to 1 upward slope with little retardation. These tests indicate that the extent and shape of excavation beyond the end of the bucket do influence the bucket dimensions. The designs improved in the order that they are numbered except for No. 1, which compared with No. 4.

The results of the study showed that the cost of any bucket would be prohibitive, so the study was discontinued.

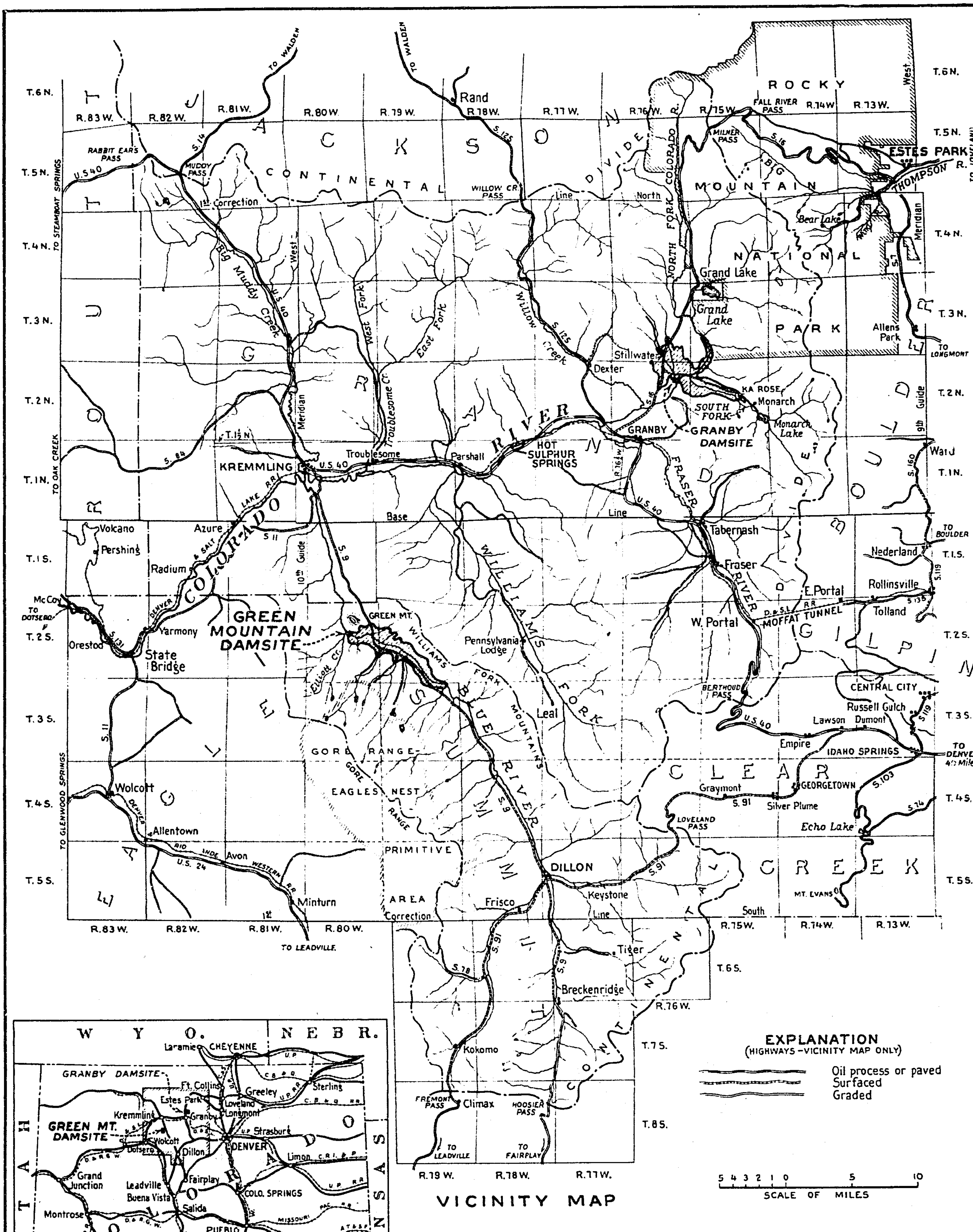
Horizontal apron stilling-basin. Model experiments were not made to determine the feasibility of this type of basin, but the dimensions for 30-, 50-, and 90-foot width basins (Figure 22D) were determined from the design chart of office memorandum to Engineer J. E. Warnock, June 8, 1938, subject, "Stilling-basin design for rectangular spillway channels." The most economical width must be determined by a cost analysis. A comparison of the required excavation for the bucket and horizontal-type basins shows very conclusively that the cost of the latter would be by far the smaller.

The above comparison brings out the conclusion that the use of each type of basin depends mainly on the tailwater conditions. The depth of water required in the bucket for satisfactory operation is considerably more than that required to form a good jump in the horizontal basin. When the height of tailwater above the riverbed is much in excess of the required d_2 jump depth, better flow conditions will exist in the bucket basin and in some cases the cost may be less. When the height of tailwater above the riverbed is less than the required d_2 jump depth, then the cost of the horizontal basin will in all cases be least.

THE OUTLET VALVES

The purpose of the outlet valve studies was to calibrate a scale model of a valve and convert the results to prototype discharges. The two prototype valves are located in the downstream end of the power house, Figure 2. Details and dimensions of the 1 to 8.333 scale model valve are shown in Figure 23. The model valve was connected to a pressure tank for accurate measurement of the head. The prototype discharge chart for one valve operating prepared from these tests is shown in Figure 24.

FIGURE I



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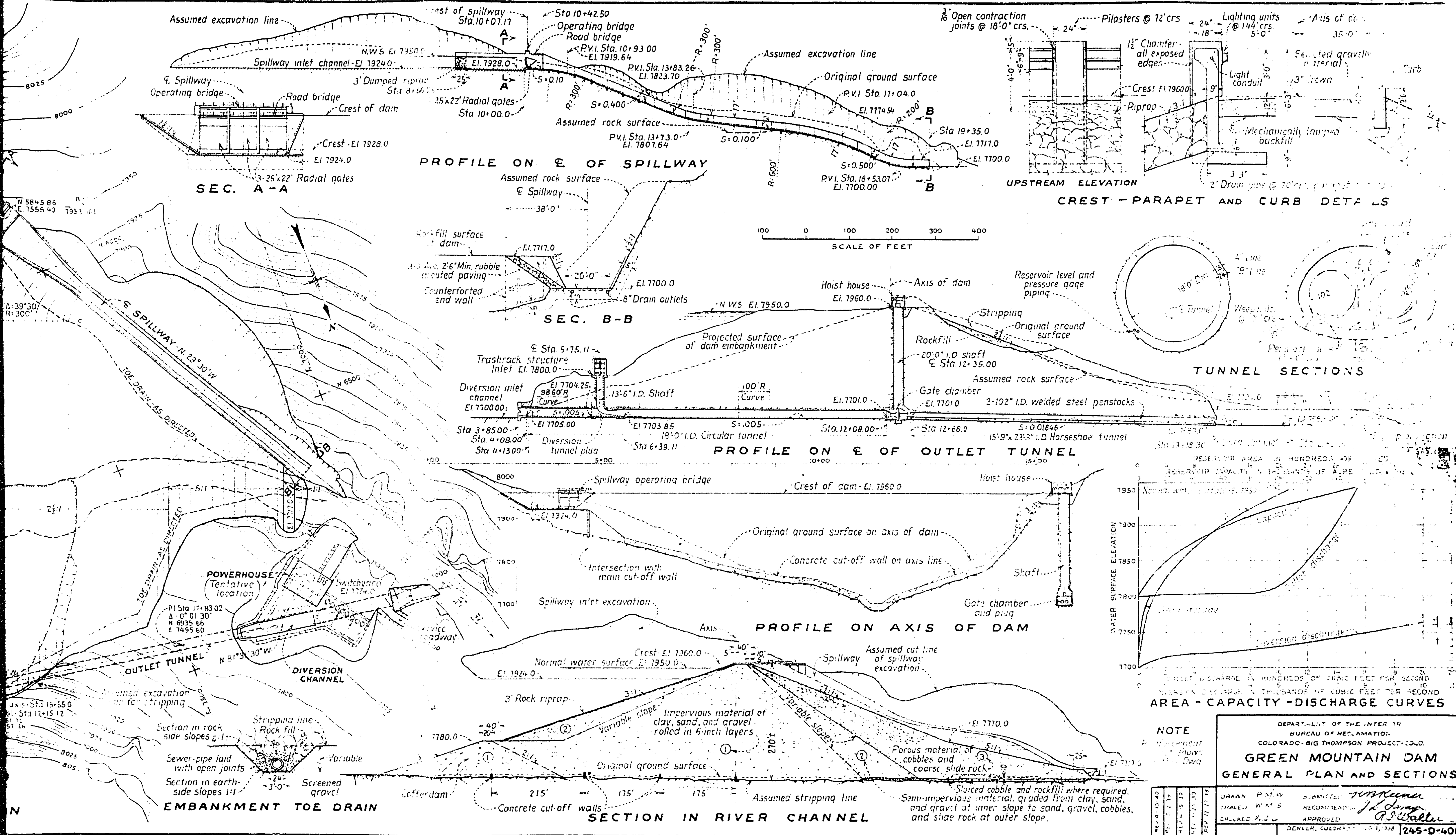
GREEN MOUNTAIN DAM LOCATION MAP

DRAWN P.M.W. SUBMITTED [signature]
TRACED W.M.S. RECOMMENDED [signature]
CHECKED J.T.M. APPROVED A. J. Walter

DENVER, COLORADO, AUG. 1, 1938 245-D-401

FIGURE 2



PROFILE ON ℄ OF SPILLWAY
SEC. A-A
SEC. B-B
CREST-PARAPET AND CURB DETAILS
UPSTREAM ELEVATION
TUNNEL SECTIONS
PROFILE ON ℄ OF OUTLET TUNNEL
PROFILE ON AXIS OF DAM
SECTION IN RIVER CHANNEL
EMBANKMENT TOE DRAIN
AREA-CAPACITY-DISCHARGE CURVES
SCALE OF FEET
POWERHOUSE (Tentative location)
OUTLET TUNNEL
DIVERSION CHANNEL
℄ SPILLWAY N. 23°30' W
TOE DRAIN AS DIRECTED
Assumed excavation line
Spillway inlet channel-El. 7924.0
Operating bridge
Road bridge
Crest of dam
3-25'x22' Radial gates
Assumed rock surface
Original ground surface
N.W.S. El. 7950.0
Hoist house El. 7960.0
Axis of dam
Trashrack structure
Diversion inlet channel
13'6" I.D. Shaft
18'0" I.D. Circular tunnel
2-102" I.D. welded steel penstocks
15'9"x 23'3" I.D. Horseshoe tunnel
Gate chamber
Rockfill
Crest of dam-El. 7960.0
Spillway operating bridge
Original ground surface on axis of dam
Concrete cut-off wall on axis line
Intersection with main cut-off wall
Spillway inlet excavation
Shaft
Gate chamber and plug
Crest-El. 7960.0
Normal water surface El. 7950.0
3' Rock riprap
Impervious material of clay, sand, and gravel rolled in 6-inch layers
Variable slope
Porous material of cobbles and coarse slide rock
Cofferdam
Concrete cut-off walls
Assumed stripping line
Sluiced cobble and rockfill where required.
Semi-impervious material, graded from clay, sand, and gravel at inner slope to sand, gravel, cobbles, and slide rock at outer slope.
Section in rock side slopes
Sewer-pipe laid with open joints
Section in earth side slopes 1:1
Screened gravel
Stripping line
Rock fill
Diversion discharge
WATER SURFACE ELEVATION
NOTE
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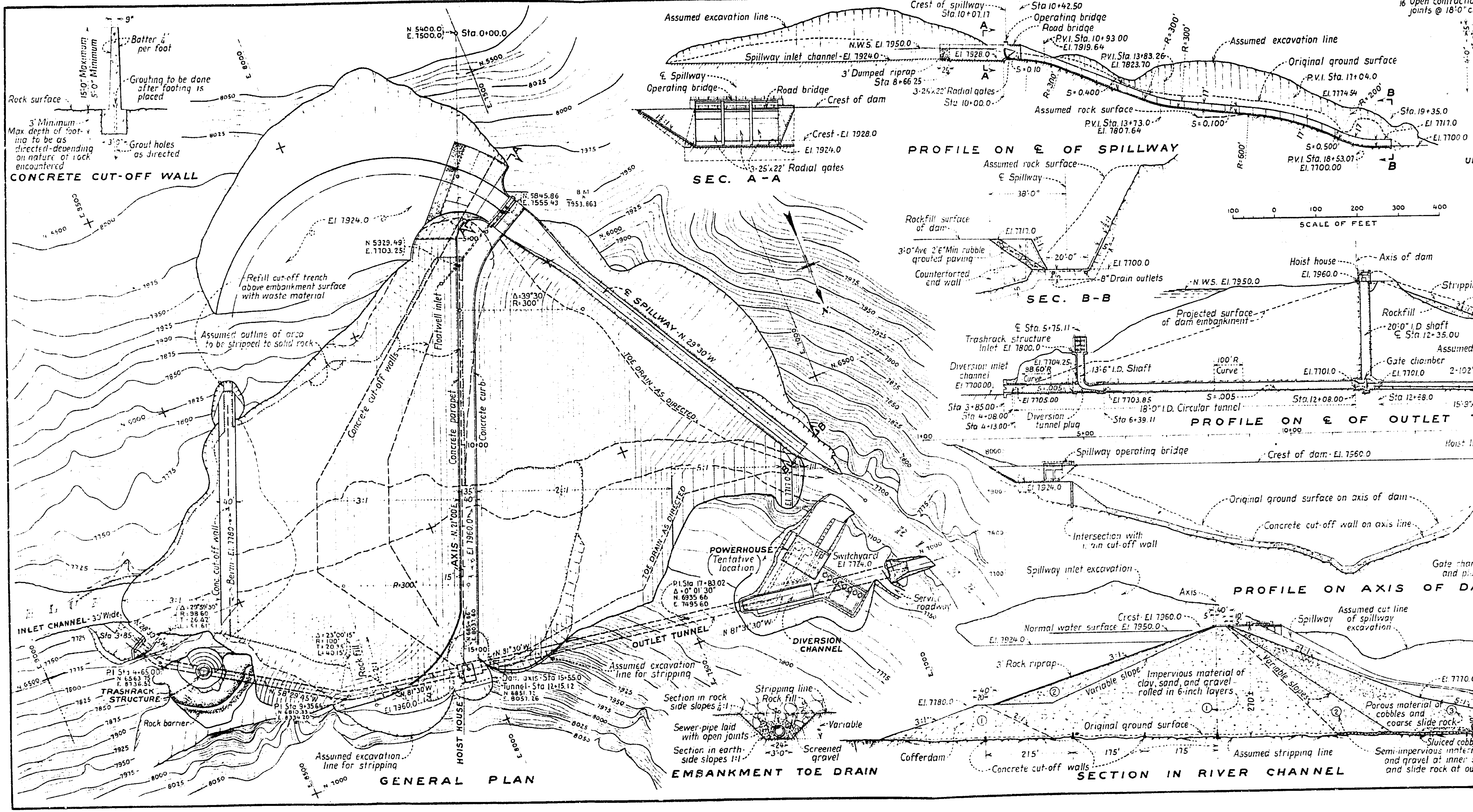
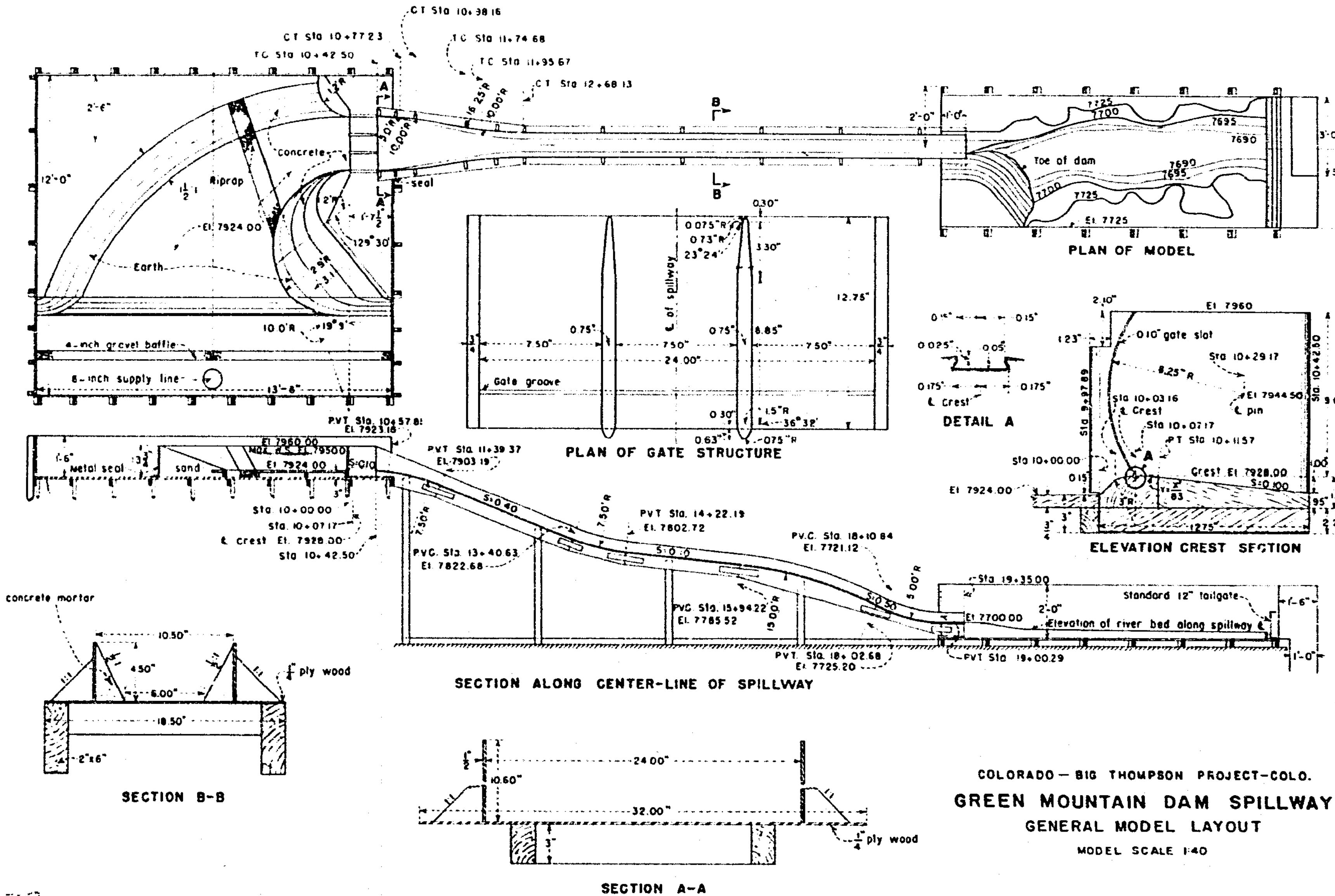
GREEN MOUNTAIN DAM
GENERAL PLAN AND SECTIONS
DRAWN
TRACED
CHECKED
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLORADO
245-D-406



CONCRETE CUT-OFF WALL
Rock surface
Batter ¼" per foot
Grouting to be done after footing is placed
Grout holes as directed
SEC. A-A
Operating bridge
Road bridge
Crest of dam
3-25'x22' Radial gates
PROFILE ON ℄ OF SPILLWAY
Assumed excavation line
Crest of spillway
Spillway inlet channel-El 1924.0
Original ground surface
Assumed rock surface
SCALE OF FEET
SEC. B-B
Countersorted end wall
8" Drain outlets
PROFILE ON ℄ OF OUTLET T
Hoist house
Axis of dam
Rockfill
Trashrack structure
13'-6" I.D. Shaft
18'-0" I.D. Circular tunnel
Diversion tunnel plug
Gate chamber
Spillway operating bridge
Original ground surface on axis of dam
Concrete cut-off wall on axis line
PROFILE ON AXIS OF DA
GENERAL PLAN
℄ SPILLWAY-N 23°30'W
TOE DRAIN AS DIRECTED
Concrete cut-off walls
Concrete parapet
Concrete curb
Floatwell inlet
AXIS-N 21°00'E
HOIST HOUSE
OUTLET TUNNEL
INLET CHANNEL-30' Wide
TRASHRACK STRUCTURE
Rock barrier
Assumed excavation line for stripping
POWERHOUSE (Tentative location)
Switchyard
Service roadway
DIVERSION CHANNEL
EMBANKMENT TOE DRAIN
Stripping line
Rock fill
Sewer-pipe laid with open joints
Screened gravel
SECTION IN RIVER CHANNEL
Impervious material of clay, sand, and gravel rolled in 6-inch layers
3' Rock riprap
Normal water surface El 1950.0
Cofferdam
Concrete cut-off walls
Assumed stripping line
Porous material of cobbles and coarse slide rock

FRAME 2



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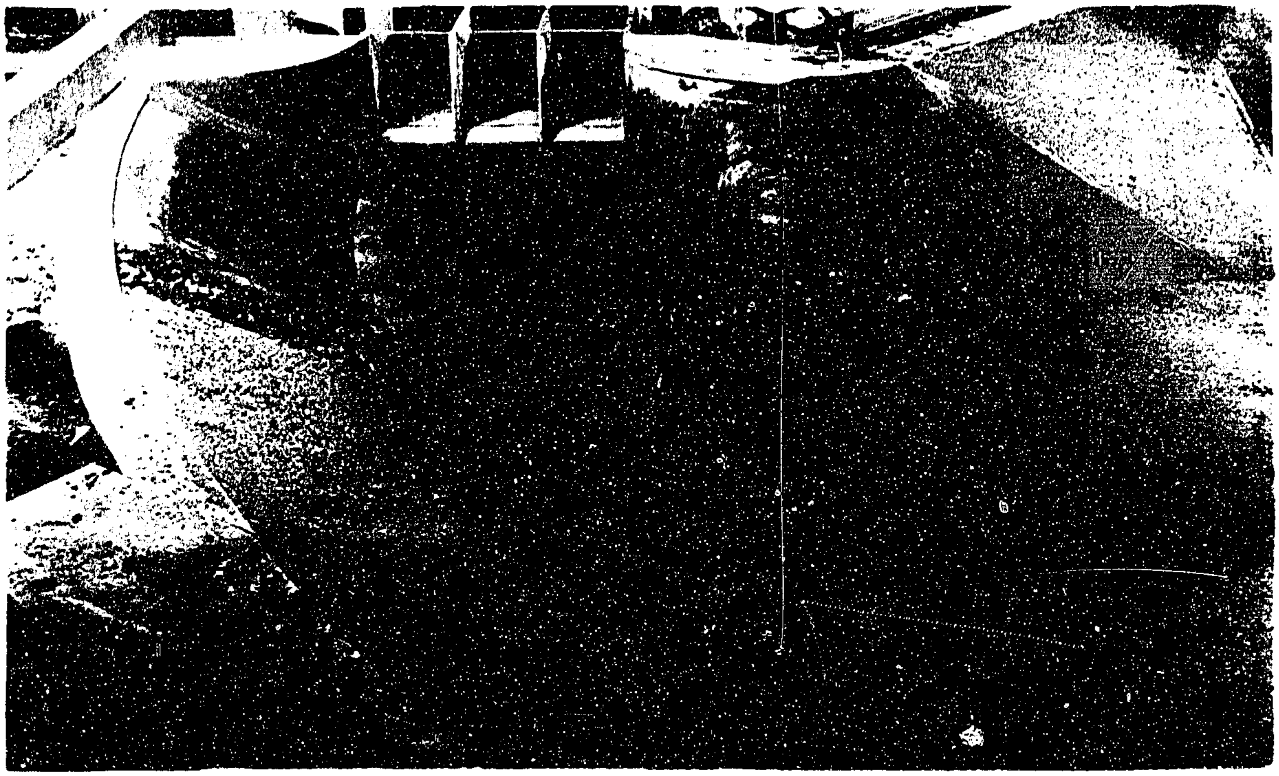
GREEN MOUNTAIN DAM SPILLWAY

GENERAL MODEL LAYOUT

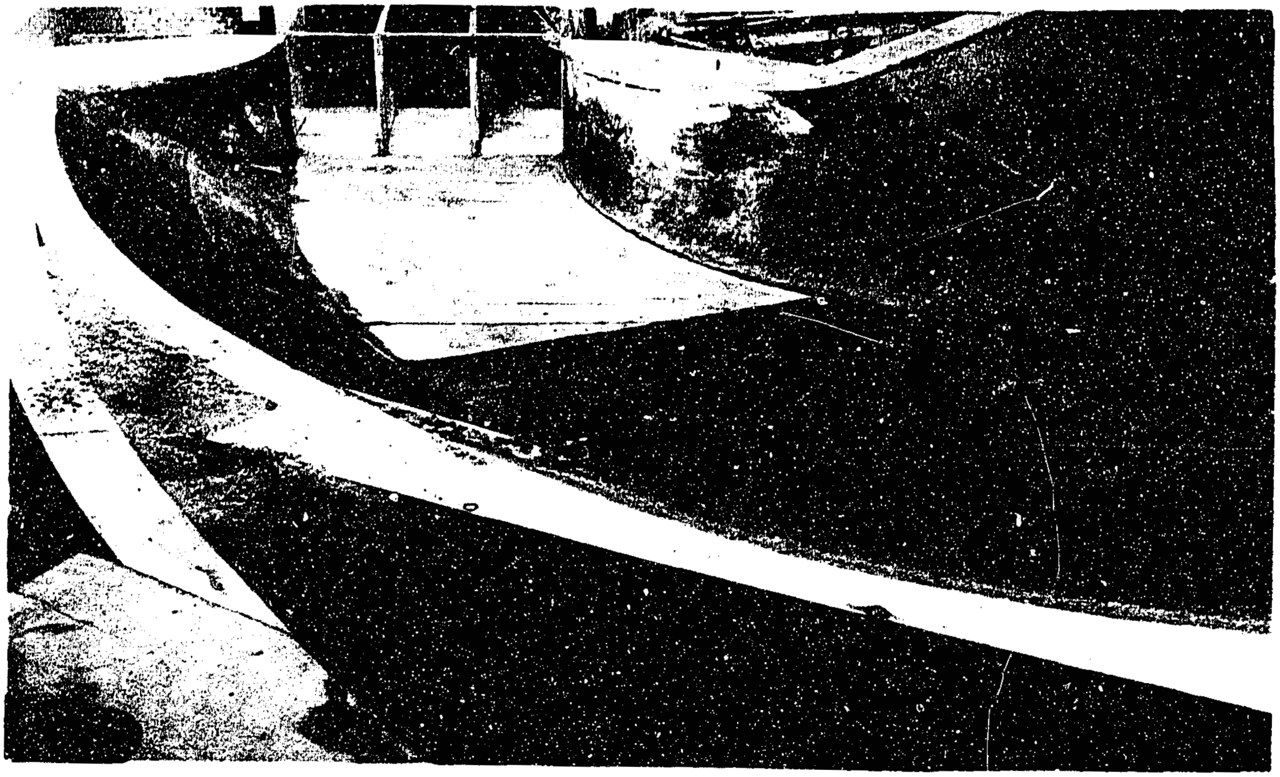
MODEL SCALE 1:40

FIGURE 3

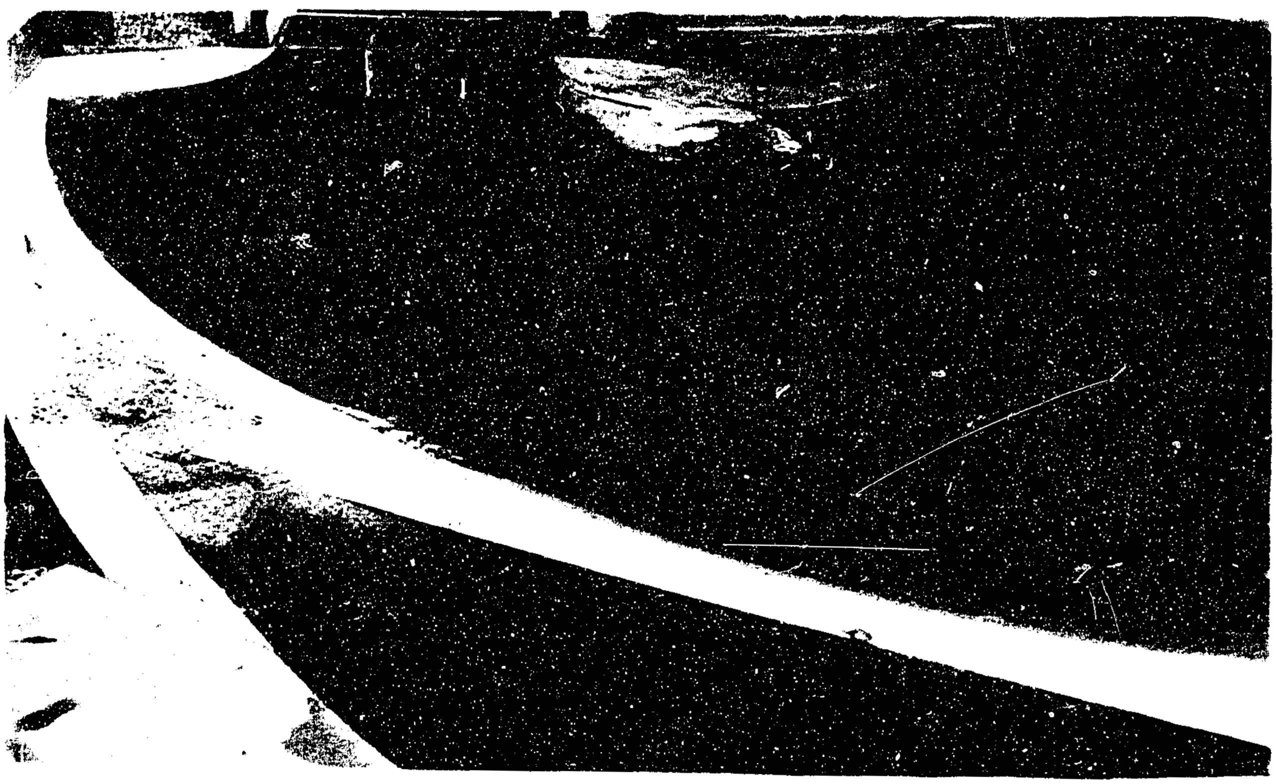


A. 400 FEET RADIUS INLET.

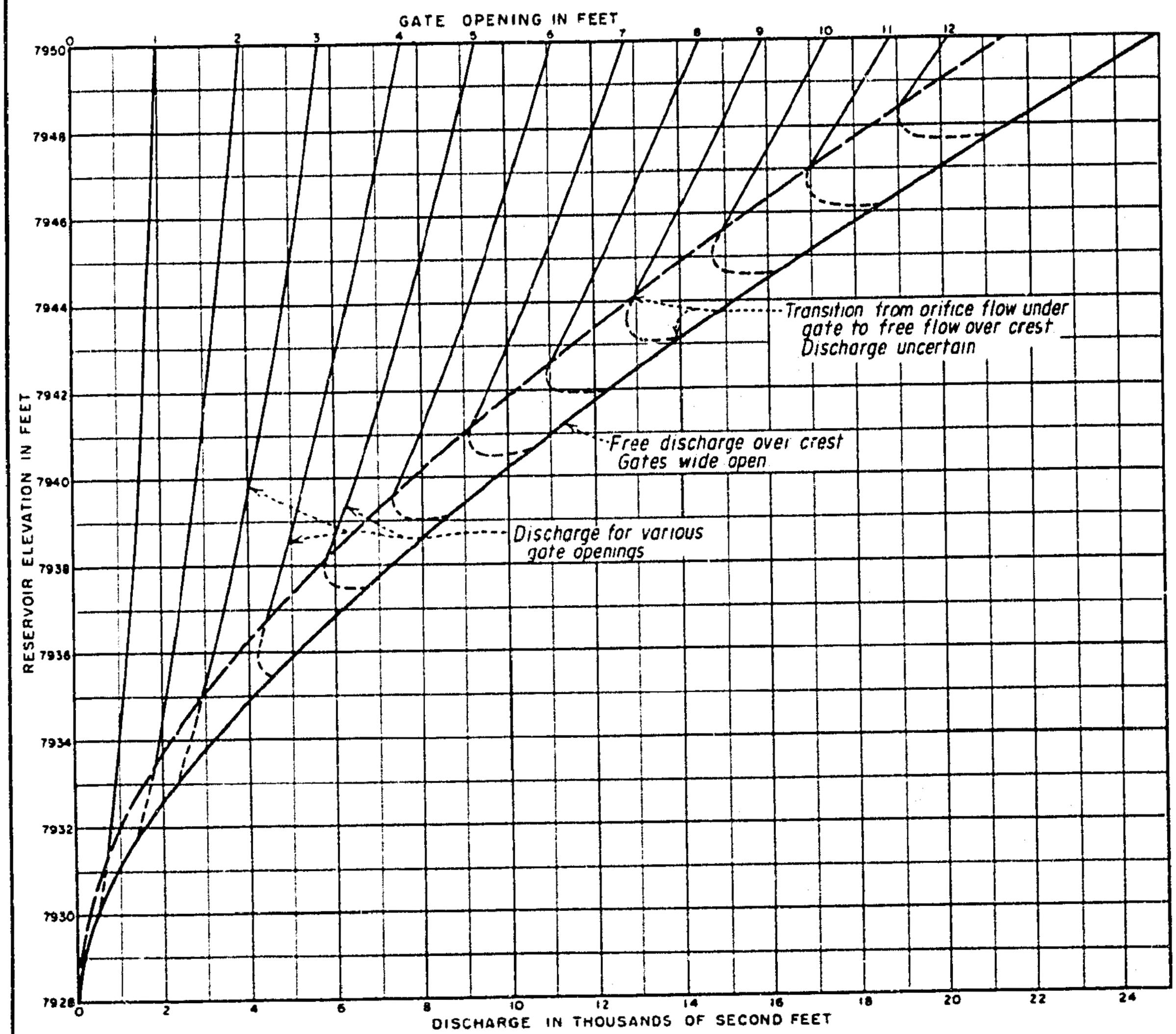
B. 400 FEET RADIUS INLET. 25,000 SECOND-FEET.



A. 200 FEET RADIUS INLET.



B. 200 FEET RADIUS INLET. 25,000 SECOND-FEET.



NOTES

1. Free discharge over crest based on experimental data from studies on Green Mountain Dam Spillway Model
2. Discharge through Gates based on Orifice formula:

$$Q = \frac{2}{3} C_o L \sqrt{2g} \left(H^{3/2} - H_T^{3/2}\right)$$

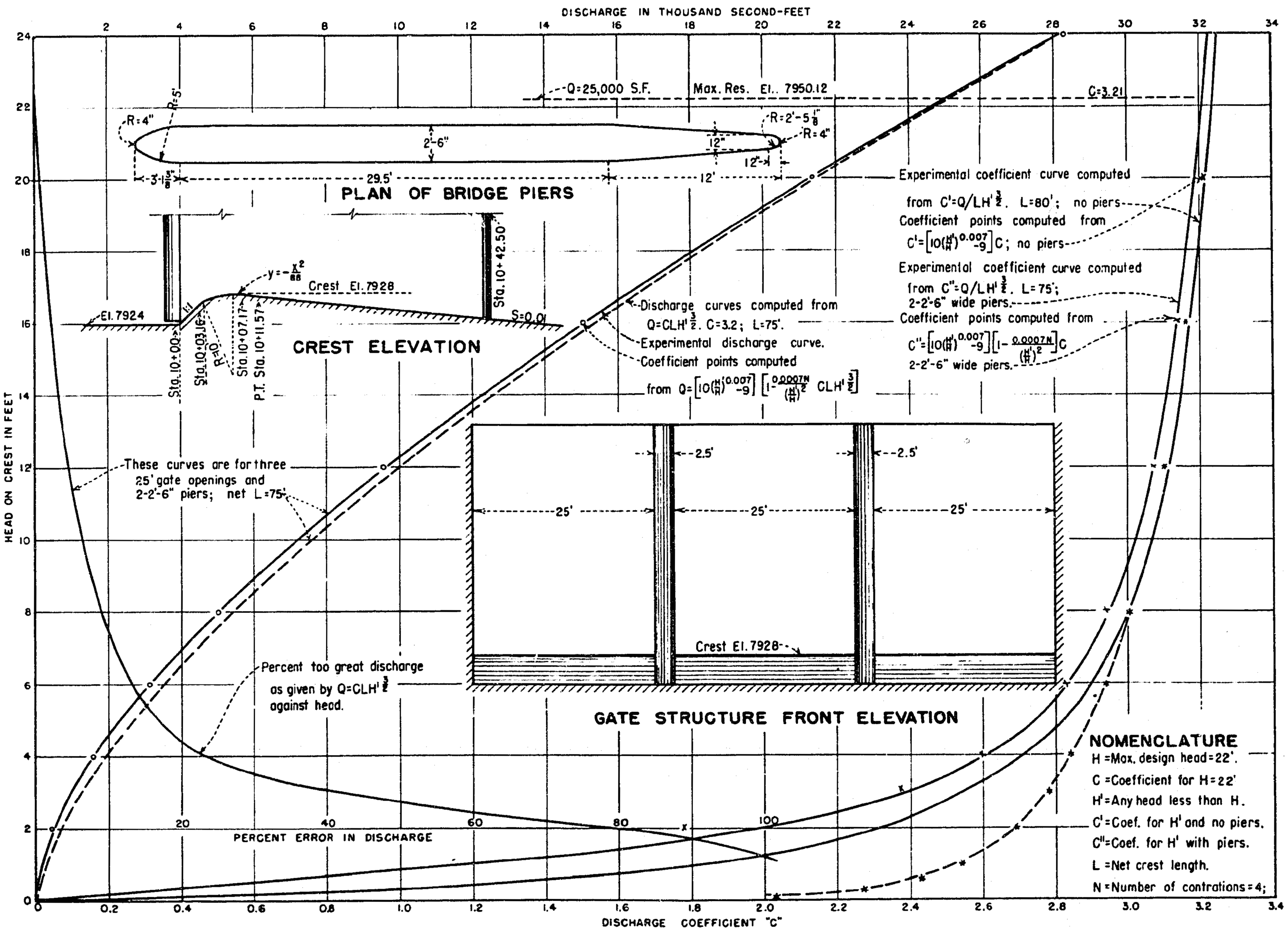
Where: Q = Discharge.
L = Crest of dam = 75'.
H = Head on crest = Res W.S. elevation − Crest elevation.
H_T = Head on bottom of gate (top of orifice).
C_o = Discharge coefficient = 0.70 based on model studies of Wheeler and Stewart Mountain dams.

GREEN MOUNTAIN DAM

DISCHARGE CURVES FOR SPILLWAY DISCHARGE

THREE GATES OPERATING AT SAME LEVEL

245-D-2285

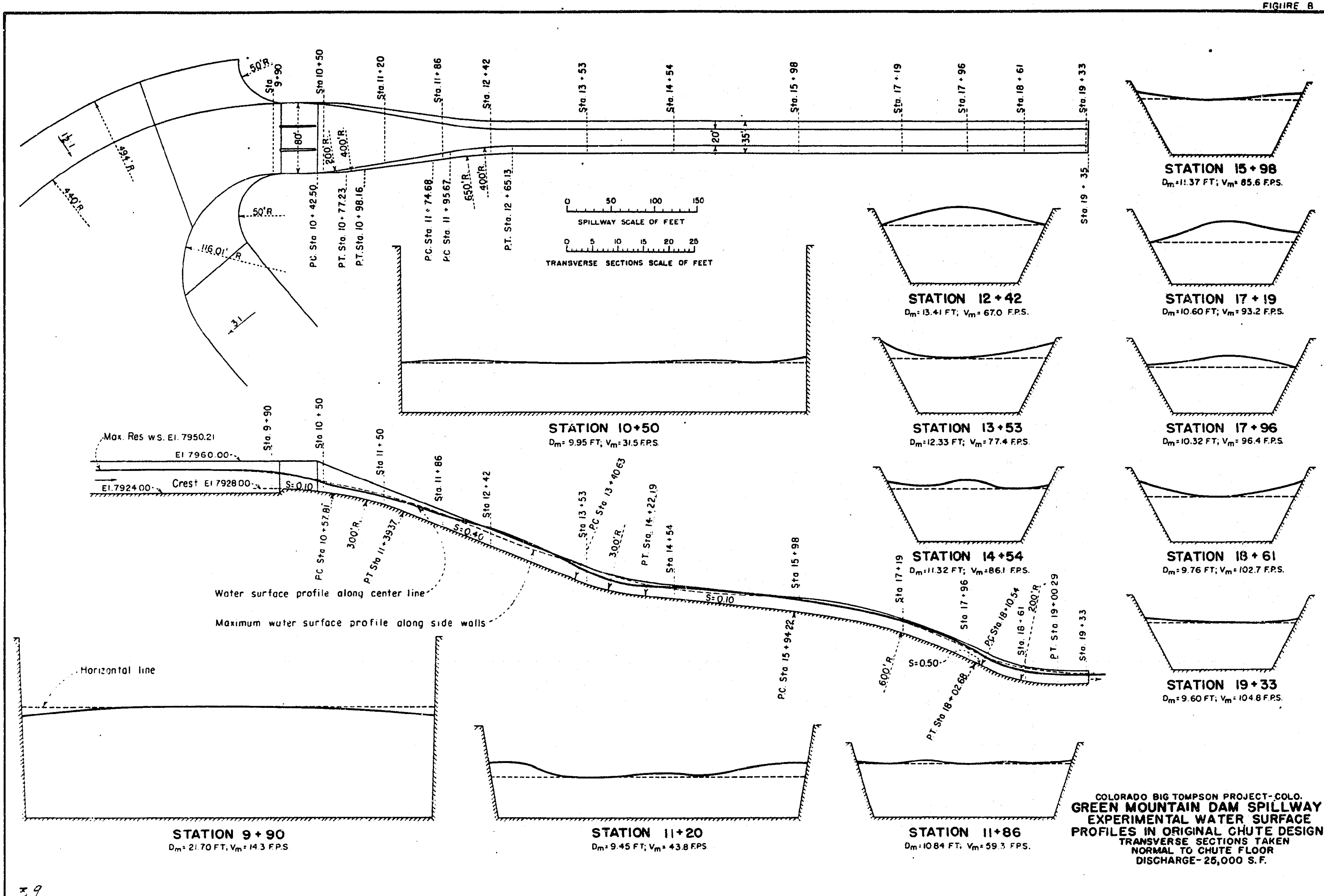


COLORADO — BIG THOMPSON PROJECT - COLO.

GREEN MOUNTAIN DAM SPILLWAY

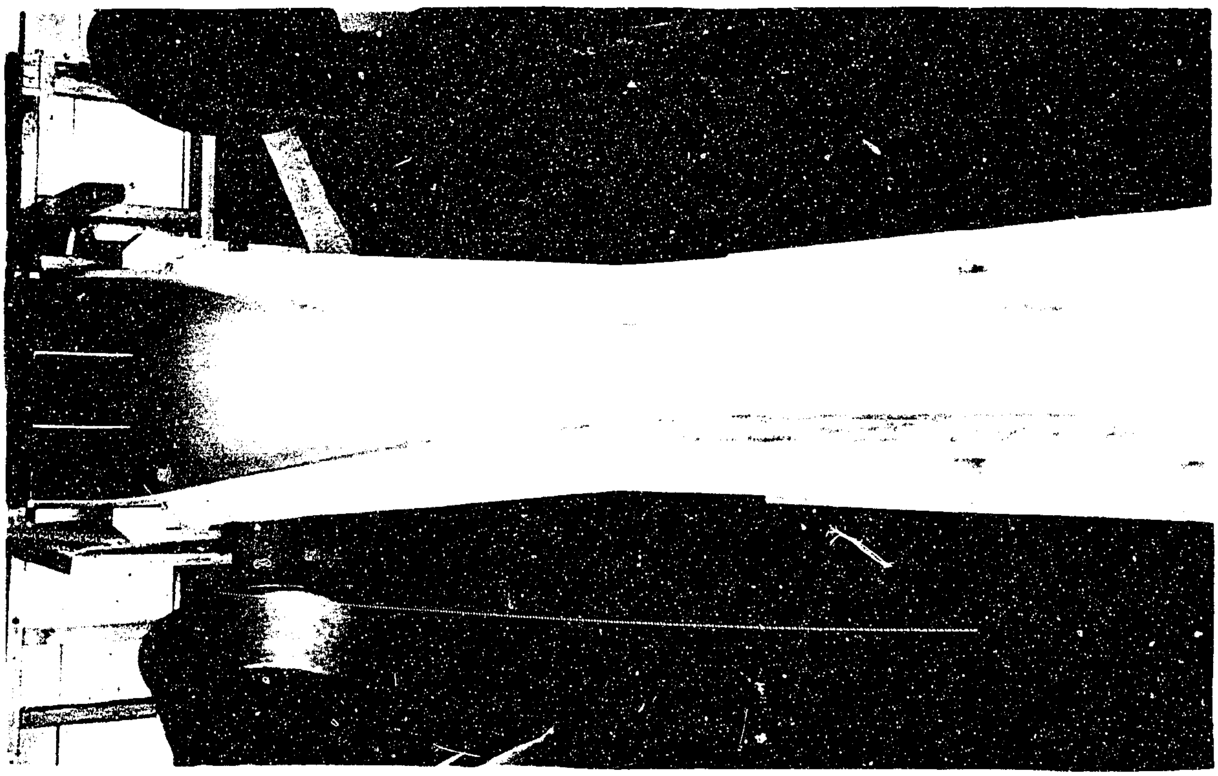
COMPARISON OF EXPERIMENTAL AND THEORETICAL SPILLWAY DISCHARGE AND COEFFICIENT CURVES

FIGURE 8

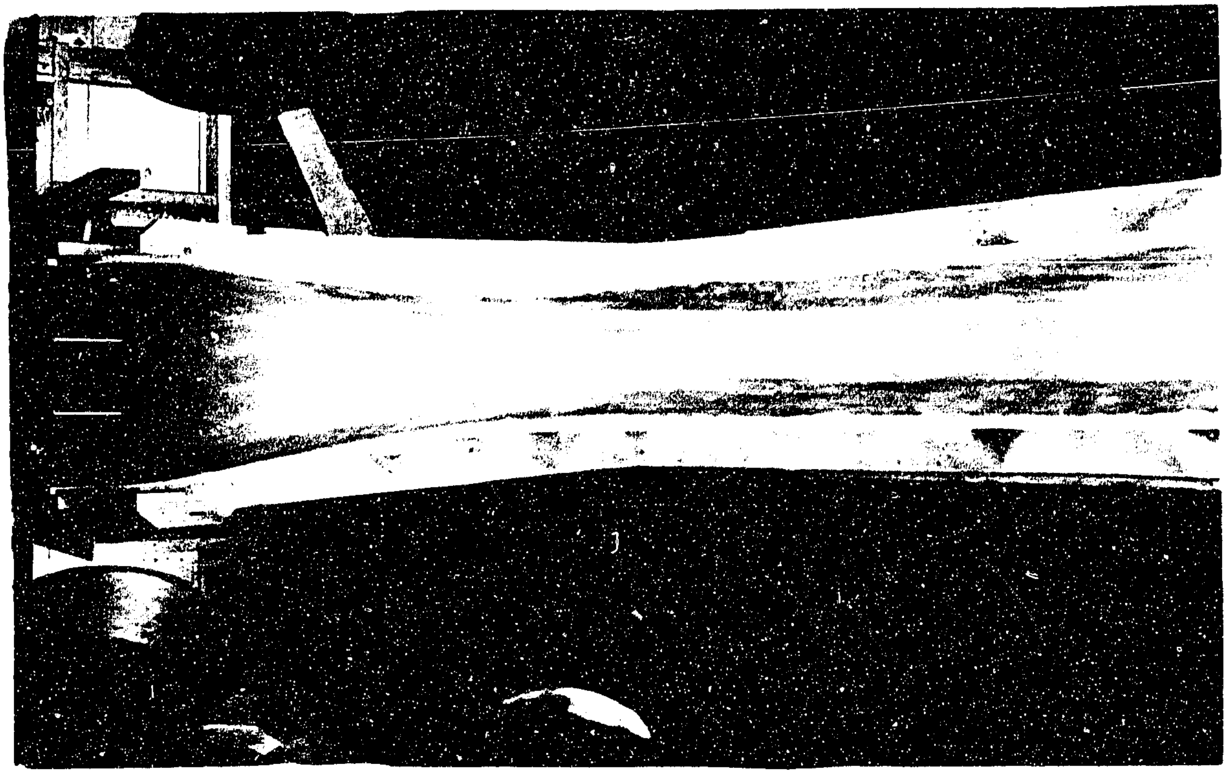


COLORADO BIG TOMPSON PROJECT-COLO.
GREEN MOUNTAIN DAM SPILLWAY
EXPERIMENTAL WATER SURFACE PROFILES IN ORIGINAL CHUTE DESIGN
TRANSVERSE SECTIONS TAKEN NORMAL TO CHUTE FLOOR
DISCHARGE-25,000 S.F.

Figure 9

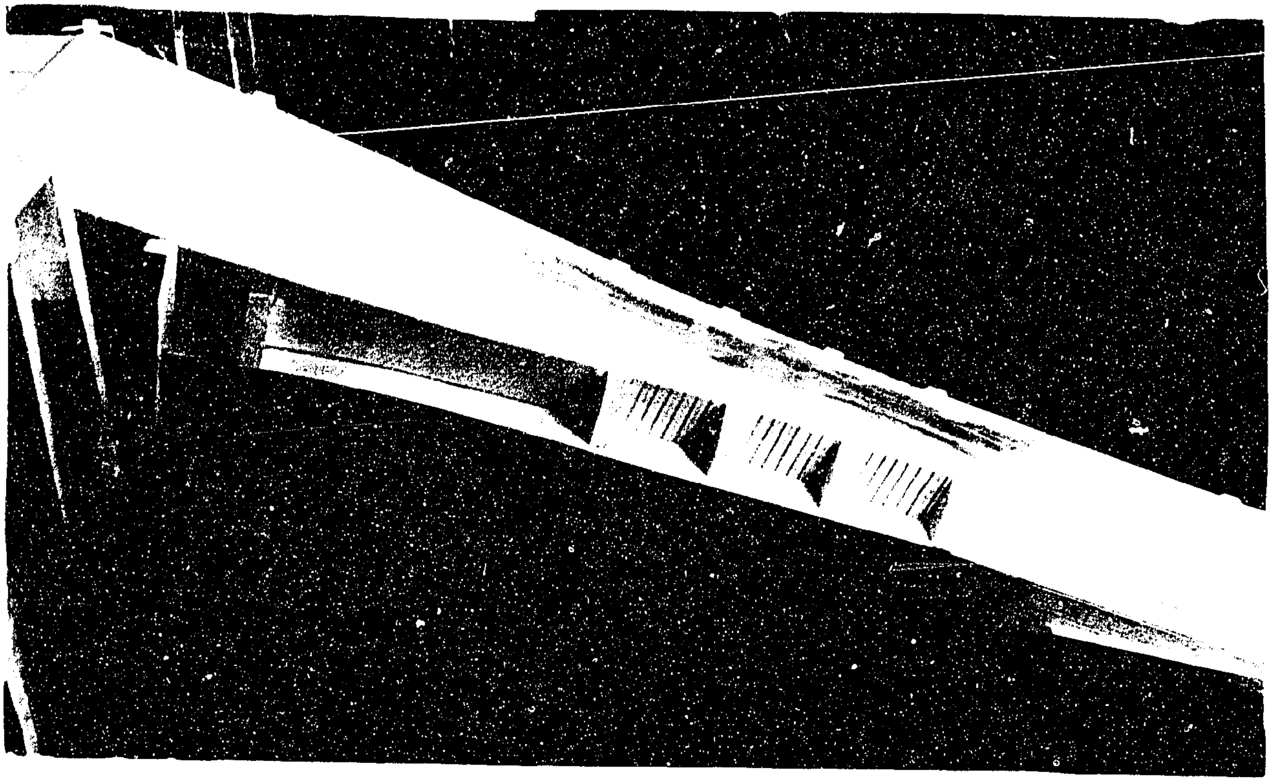


A. ORIGINAL CHUTE DESIGN.

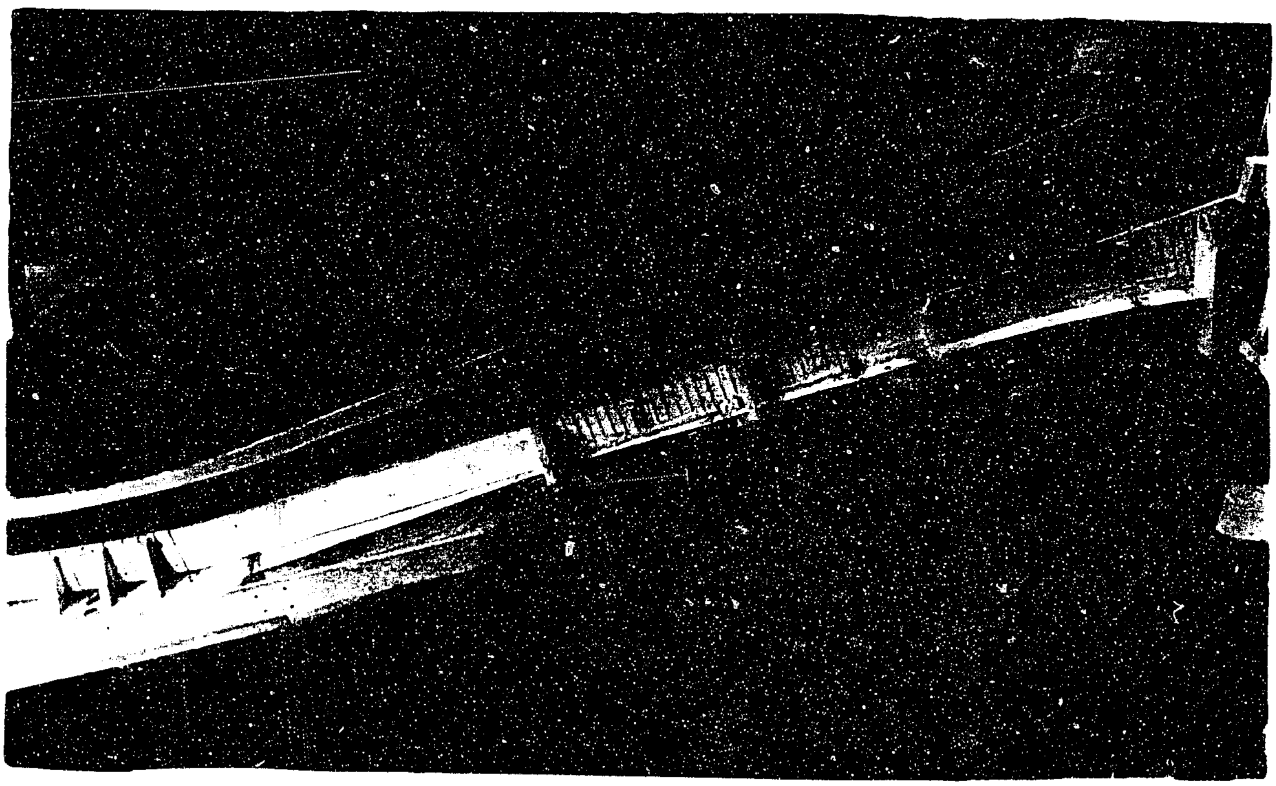


B. ORIGINAL CHUTE DESIGN. 25,000 SECOND-FEET.

Figure 10

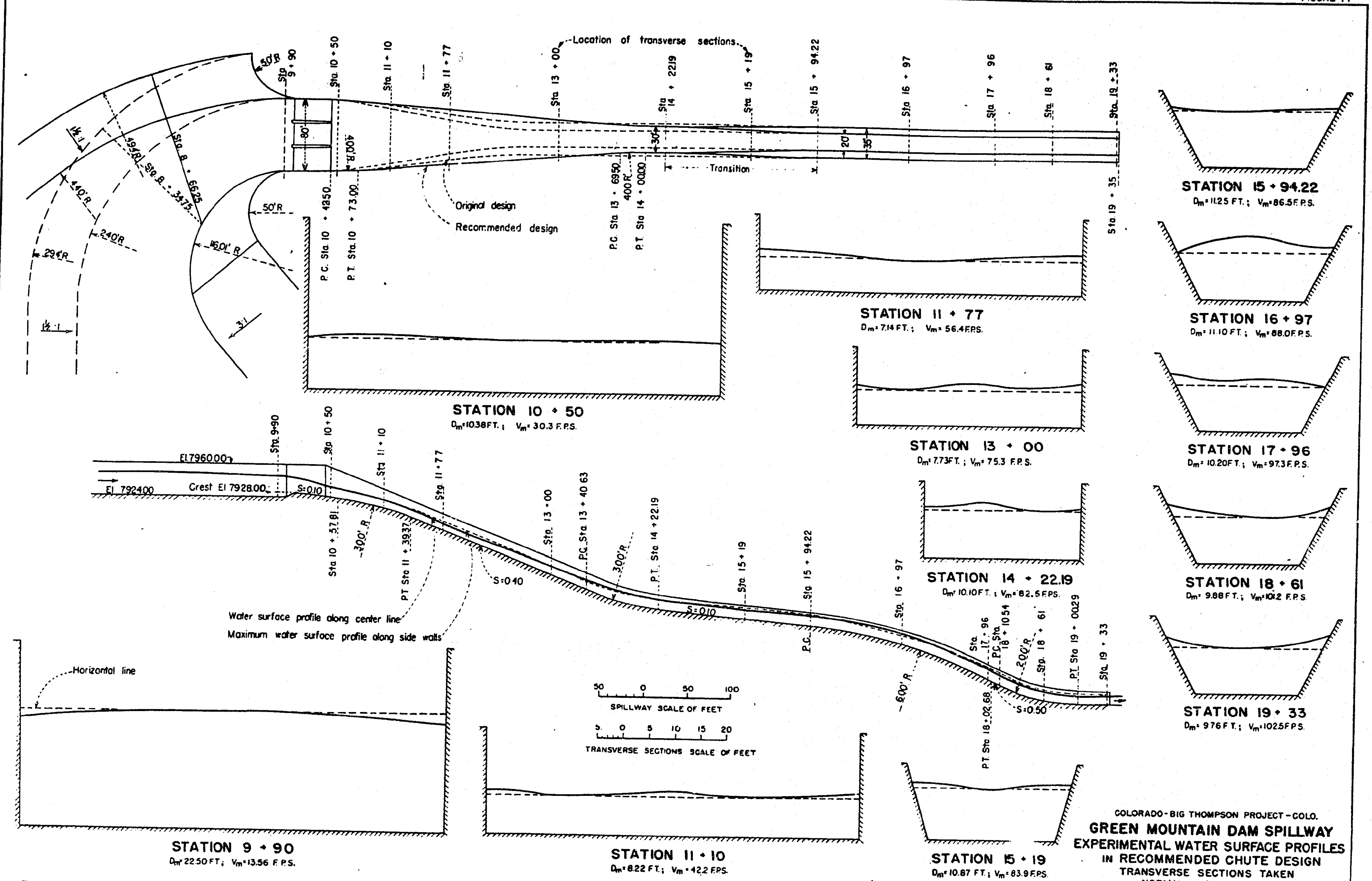


A. ORIGINAL CHUTE DESIGN. 25,000 SECOND-FEET.



B. RECOMMENDED CHUTE DESIGN. 25,000 SECOND-FEET (COLORED WATER).

FIGURE 11



COLORADO-BIG THOMPSON PROJECT-COLO.

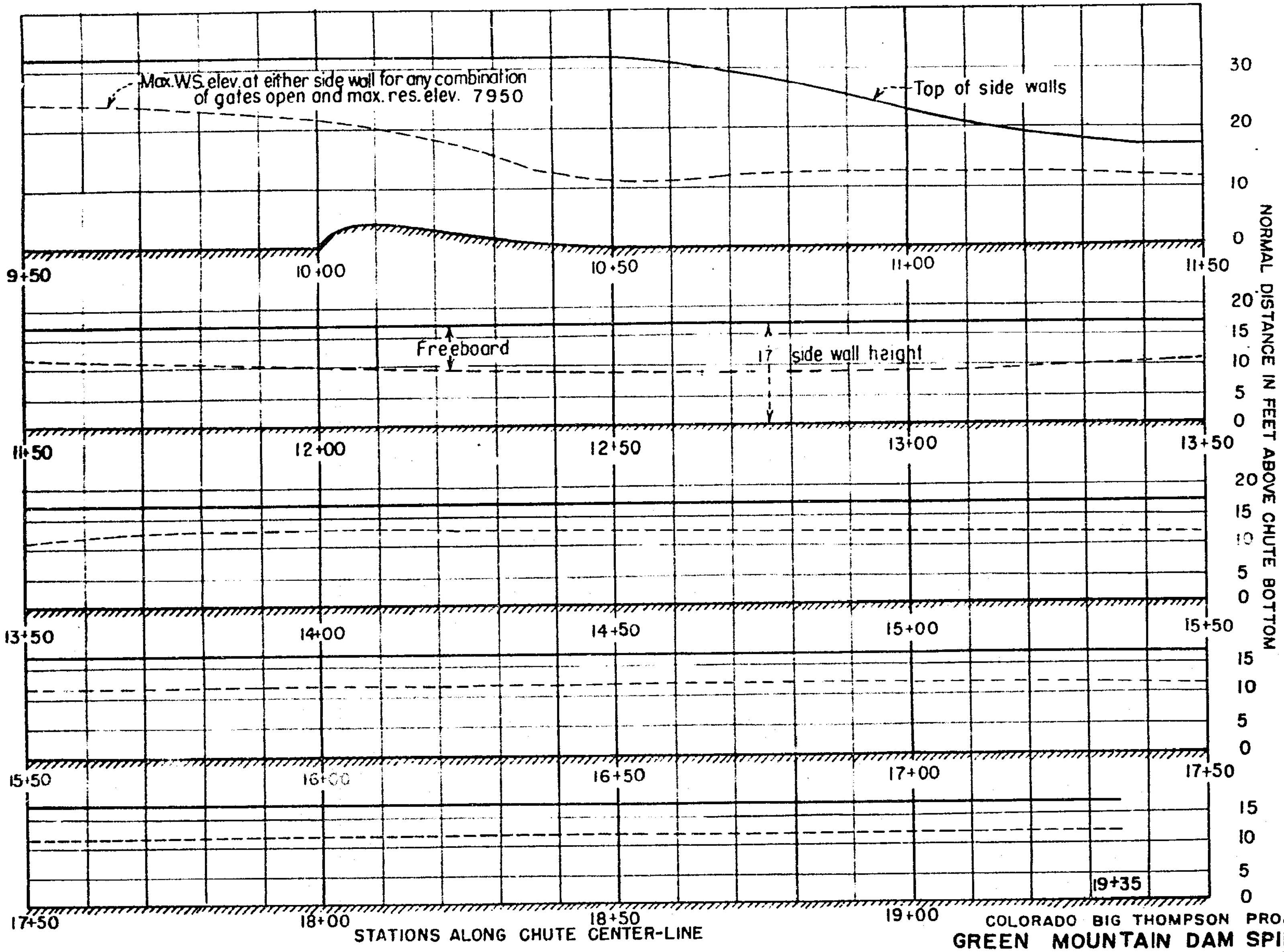
GREEN MOUNTAIN DAM SPILLWAY

EXPERIMENTAL WATER SURFACE PROFILES IN RECOMMENDED CHUTE DESIGN
TRANSVERSE SECTIONS TAKEN NORMAL TO CHUTE FLOOR
DISCHARGE - 25,000 S.F.

Figure 12

A. RECOMMENDED CHUTE DESIGN

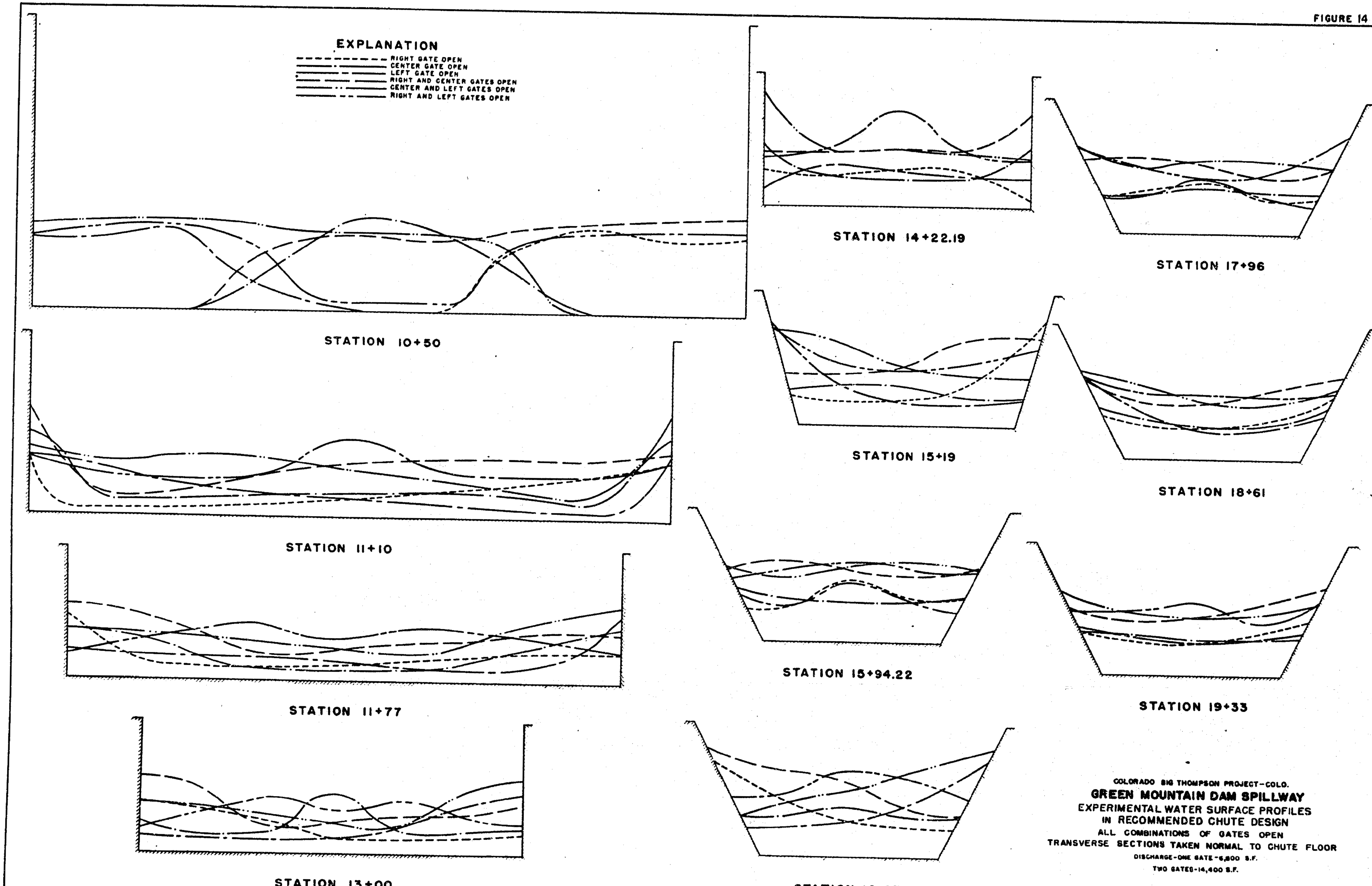
B. RECOMMENDED CHUTE DESIGN. 25,000 SECOND-FEET (COLORED WATER).



COLORADO BIG THOMPSON PROJECT
GREEN MOUNTAIN DAM SPILLWAY
MAXIMUM WATER SURFACE ELEVATION AT SIDE WALLS OF CHUTE

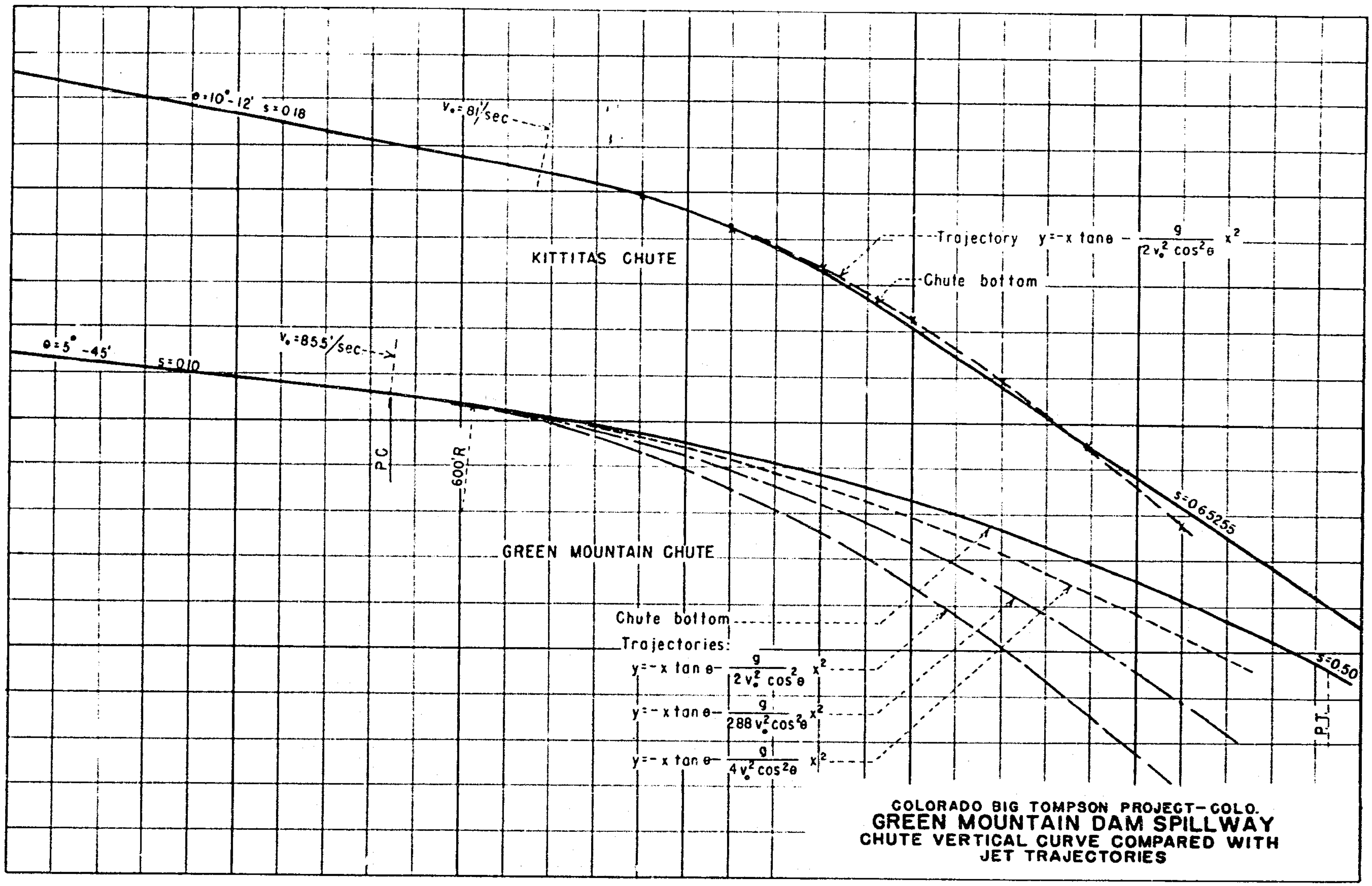
FIGURE 13

FIGURE 14

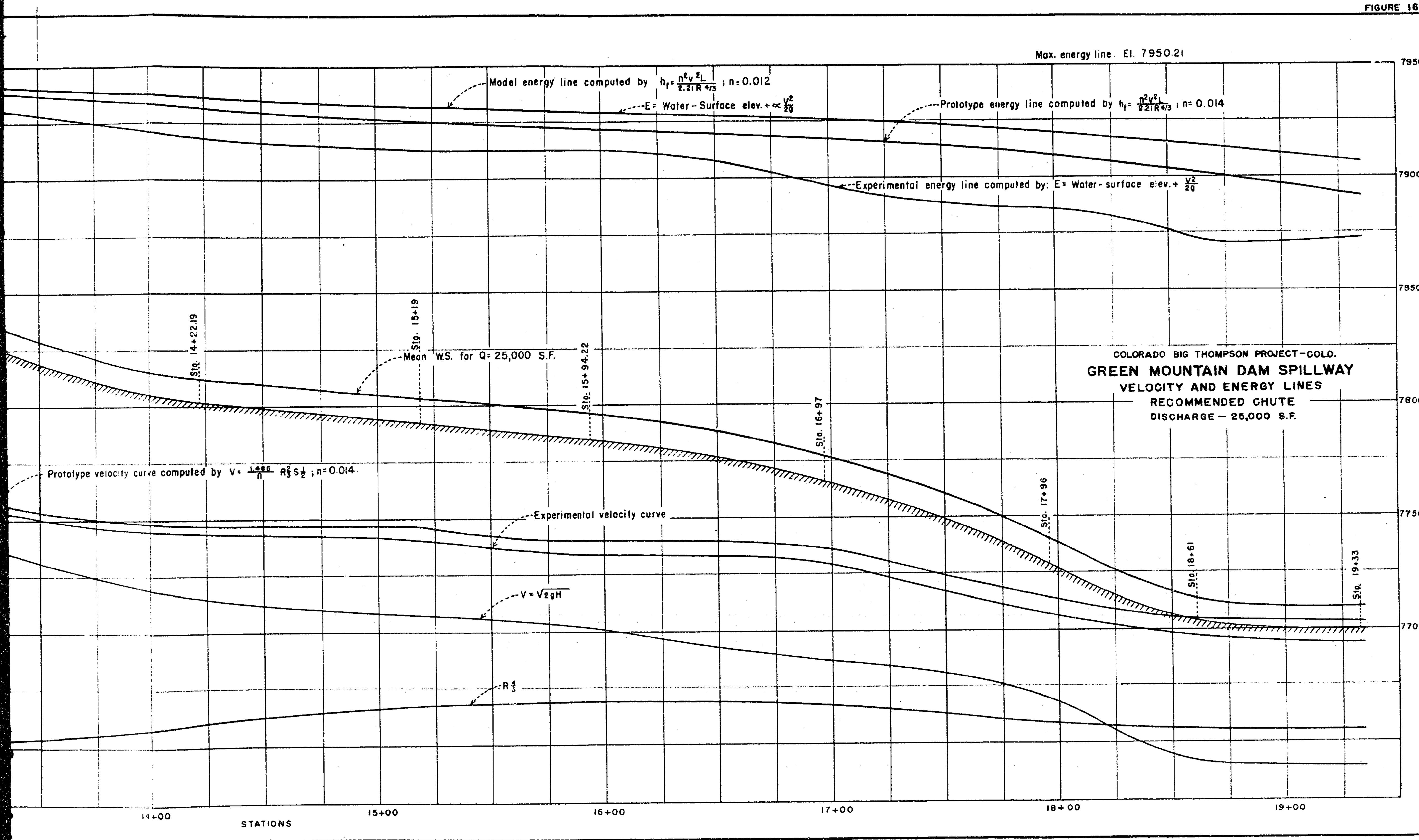


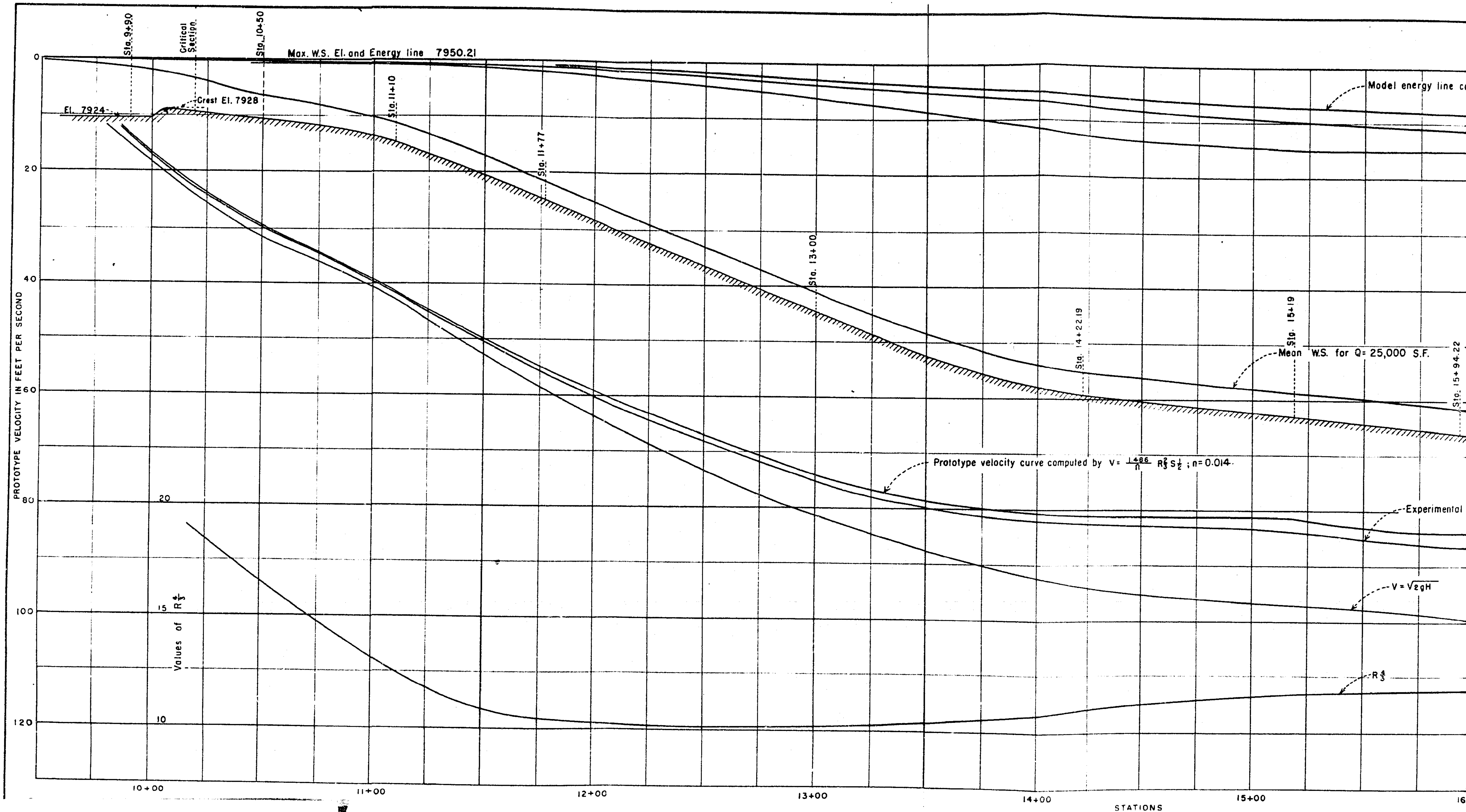
COLORADO BIG THOMPSON PROJECT—COLO.
GREEN MOUNTAIN DAM SPILLWAY
EXPERIMENTAL WATER SURFACE PROFILES
IN RECOMMENDED CHUTE DESIGN
ALL COMBINATIONS OF GATES OPEN
TRANSVERSE SECTIONS TAKEN NORMAL TO CHUTE FLOOR
DISCHARGE-ONE GATE-6,800 S.F.
TWO GATES-14,400 S.F.

FIGURE 15



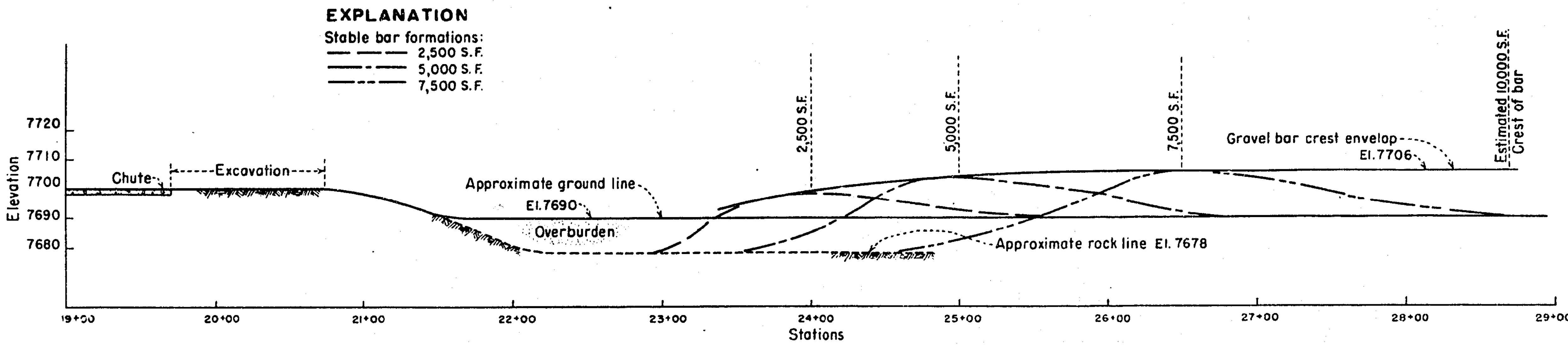
COLORADO BIG TOMPSON PROJECT-COLO.
GREEN MOUNTAIN DAM SPILLWAY
CHUTE VERTICAL CURVE COMPARED WITH
JET TRAJECTORIES


Max. energy line El. 7950.21
Model energy line computed by $h_f = \frac{n^2 v^2 L}{2.21 R^{4/3}}$; n = 0.012
E = Water-Surface elev. + $\propto \frac{V^2}{2g}$
Prototype energy line computed by $h_f = \frac{n^2 v^2 L}{2.21 R^{4/3}}$; n = 0.014
Experimental energy line computed by: E = Water-surface elev. + $\frac{V^2}{2g}$
Mean W.S. for Q = 25,000 S.F.
Sta. 14+22.19
Sta. 15+19
Sta. 15+94.22
Sta. 16+97
Sta. 17+96
Sta. 18+61
Sta. 19+33
Prototype velocity curve computed by $V = \frac{1.486}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$; n = 0.014
Experimental velocity curve
$V = \sqrt{2gH}$
$R^{\frac{4}{3}}$
COLORADO BIG THOMPSON PROJECT-COLO.
GREEN MOUNTAIN DAM SPILLWAY
VELOCITY AND ENERGY LINES
RECOMMENDED CHUTE
DISCHARGE – 25,000 S.F.
7950
7900
7850
7800
7750
7700
PROTOTYPE ELEVATION
14+00
15+00
16+00
17+00
18+00
19+00
STATIONS

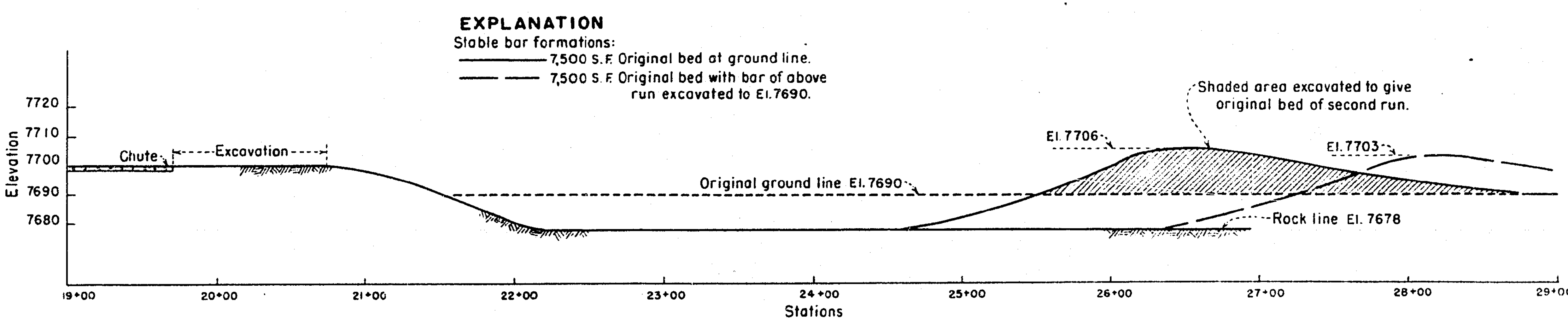


Sta. 9+90
Critical Section
Sta. 10+50
Max. W.S. El. and Energy line 7950.21
Model energy line co
El. 7924
Crest El. 7928
Sta. 11+10
Sta. 11+77
Sta. 13+00
Sta. 14+22.19
Sta. 15+19
Sta. 15+94.22
Mean W.S. for Q= 25,000 S.F.
Prototype velocity curve computed by $V=\frac{1.486}{n}R^{\frac{2}{3}}S^{\frac{1}{2}}$; n=0.014
Experimental v
$V=\sqrt{2gH}$
$R^{\frac{4}{3}}$
Values of $R^{\frac{4}{3}}$
PROTOTYPE VELOCITY IN FEET PER SECOND
STATIONS

FRAME 2



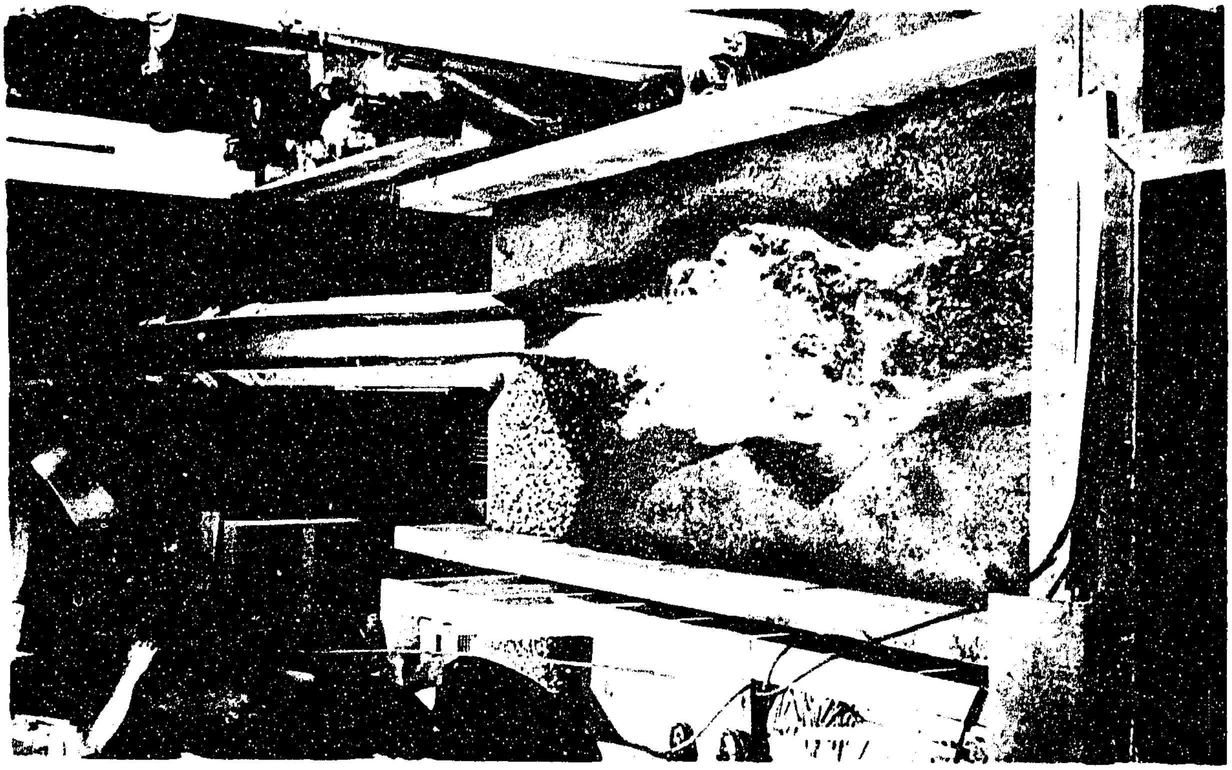
A. CENTER-LINE PROFILE OF GRAVEL BAR FORMATIONS FOR VARIOUS DISCHARGES



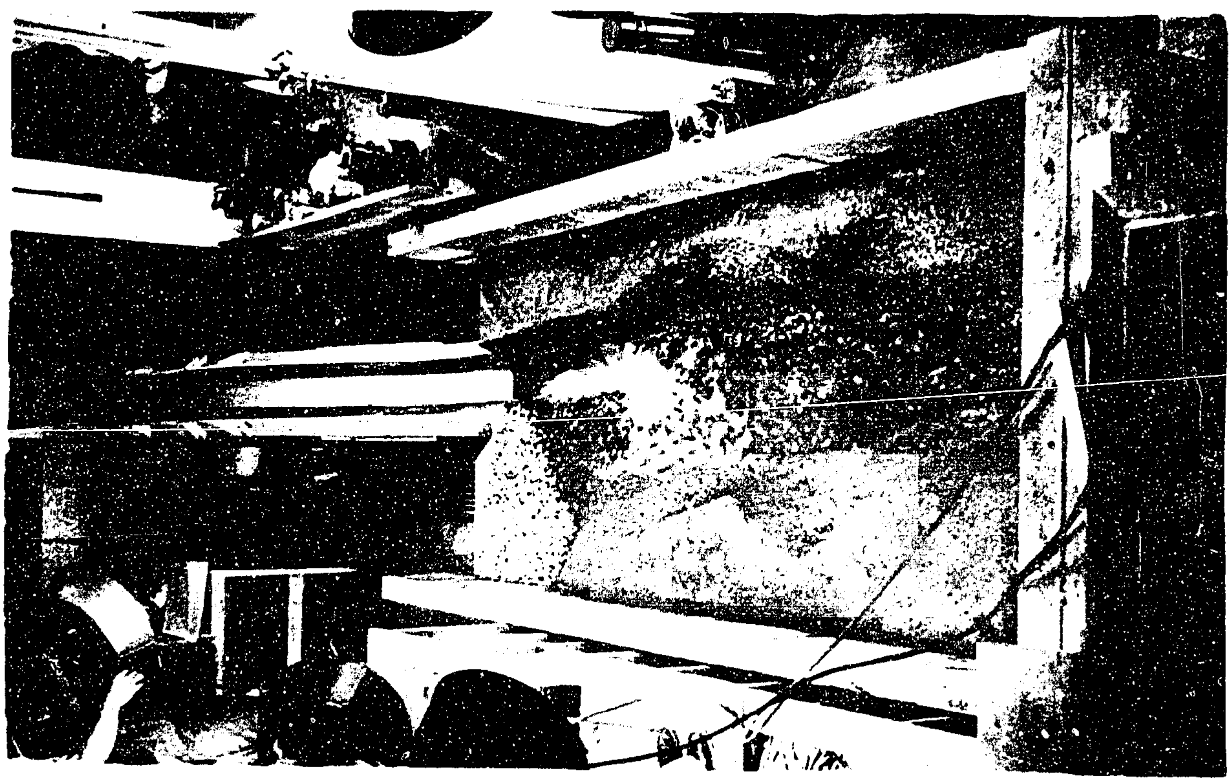
B. GRAVEL BAR FORMED BY A SECOND FLOOD AFTER THE BAR RESULTING FROM A FIRST FLOOD IS EXCAVATED TO THE ORIGINAL RIVER BED

COLORADO BIG THOMPSON PROJECT—COLO.
GREEN MOUNTAIN DAM SPILLWAY
SCOUR STUDIES

Figure 18

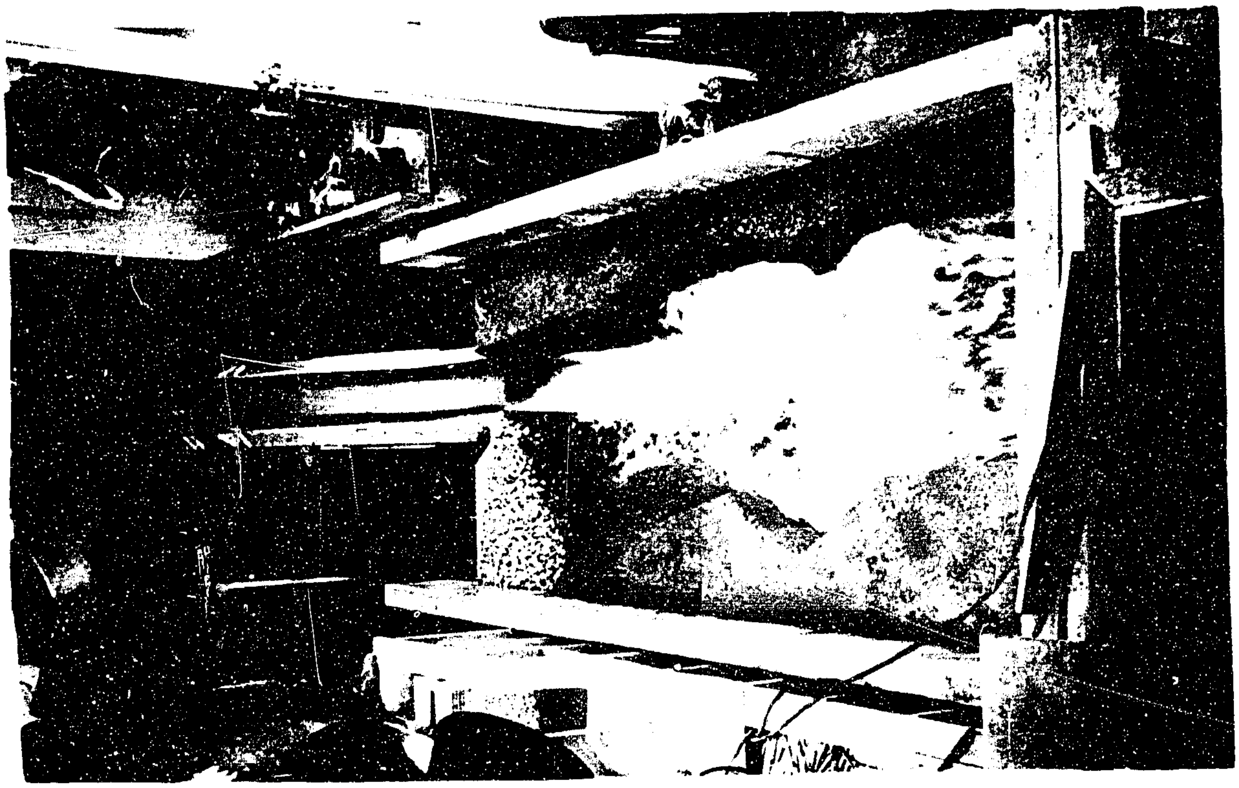


B. 5,000 SECOND-FEET.

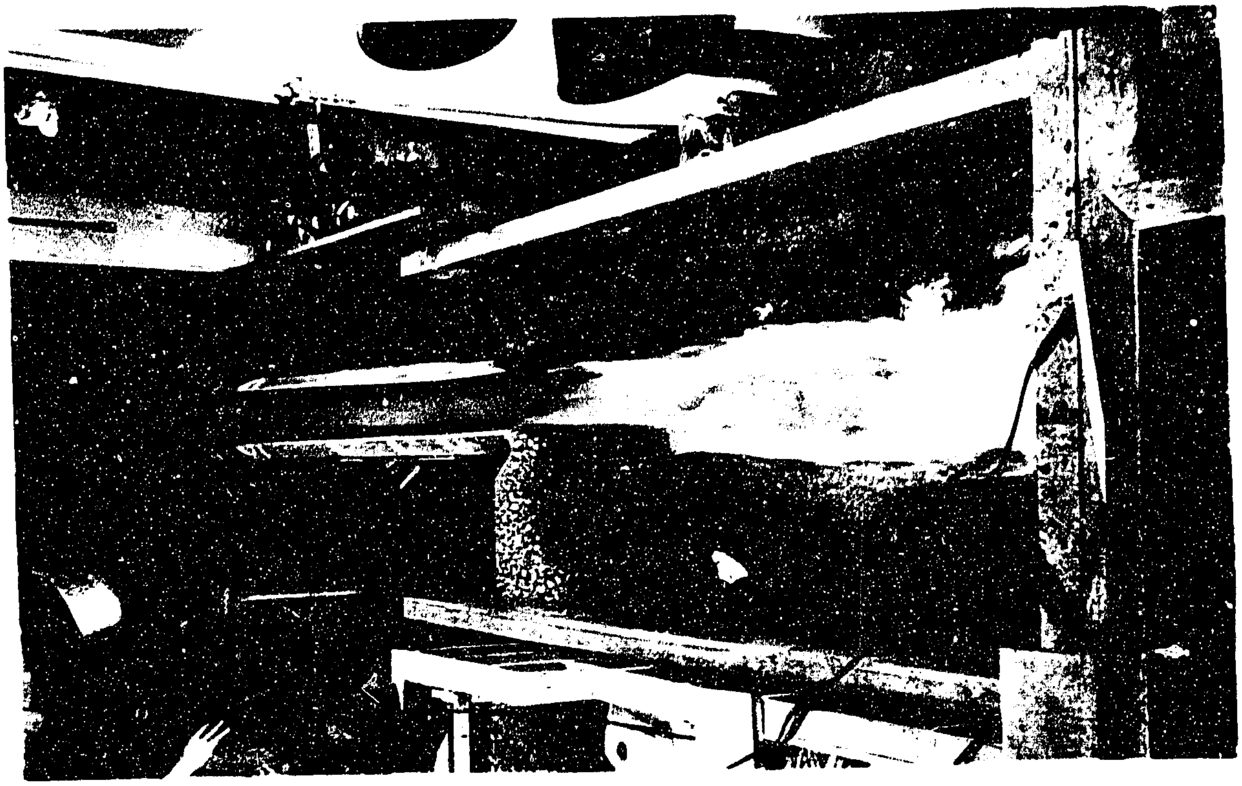


A. 2,500 SECOND-FEET

Figure 19



A. 7,500 SECOND-FEET.



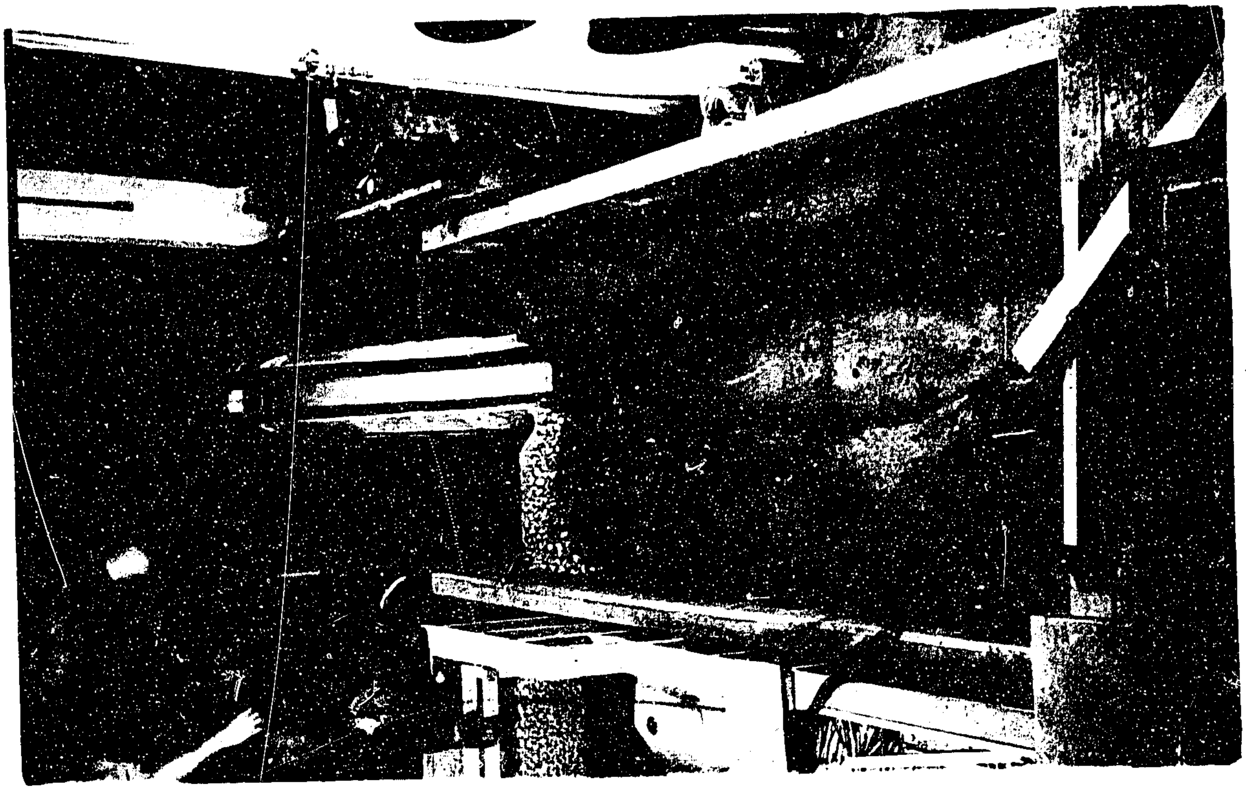
B. 10,000 SECOND-FEET.

Figure 20

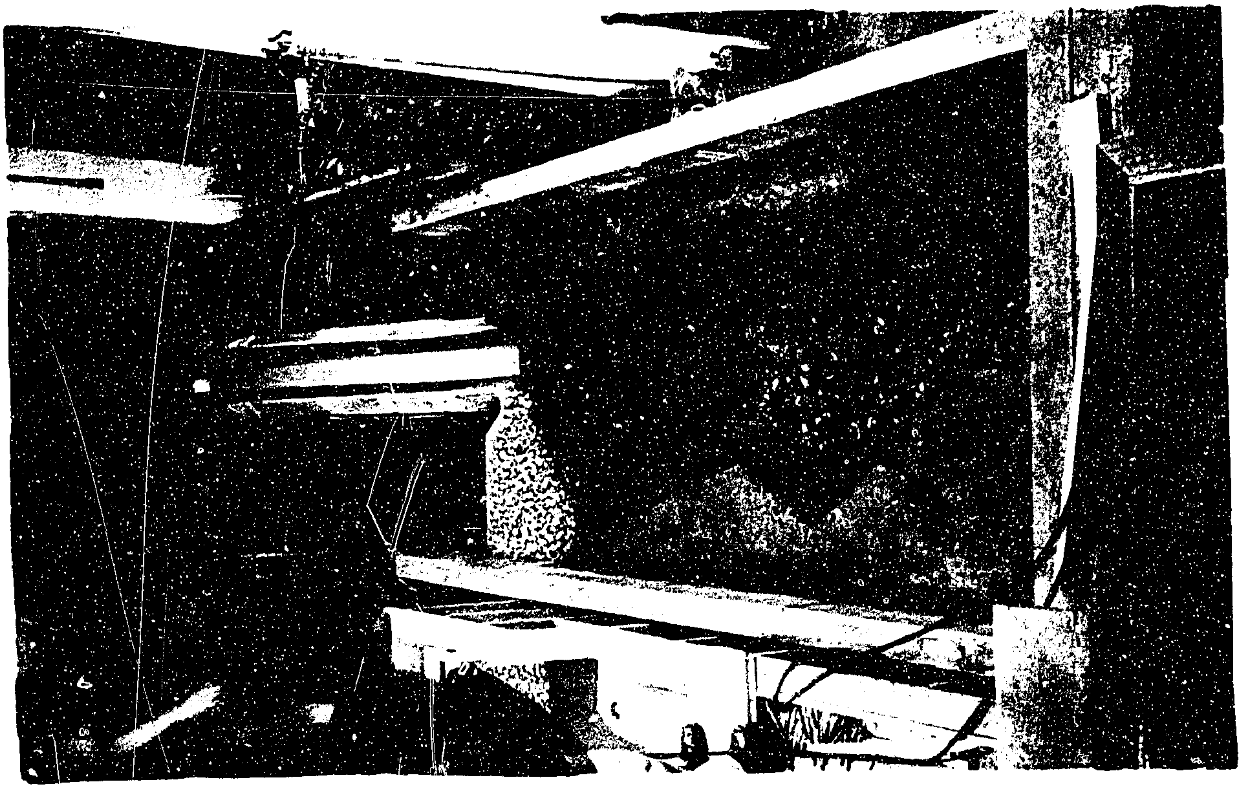
A. ORIGINAL BED FOR EACH RUN.

B. SCOUR FOR 2.500 SECOND-FEET.

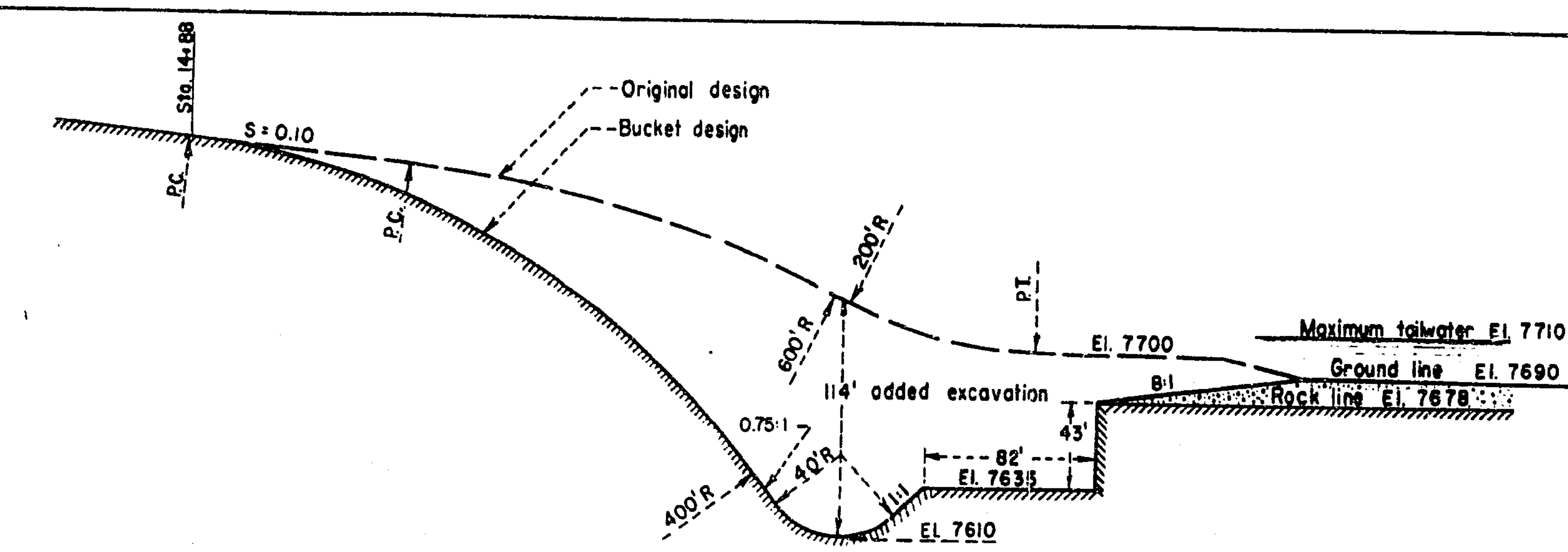
Figure 21



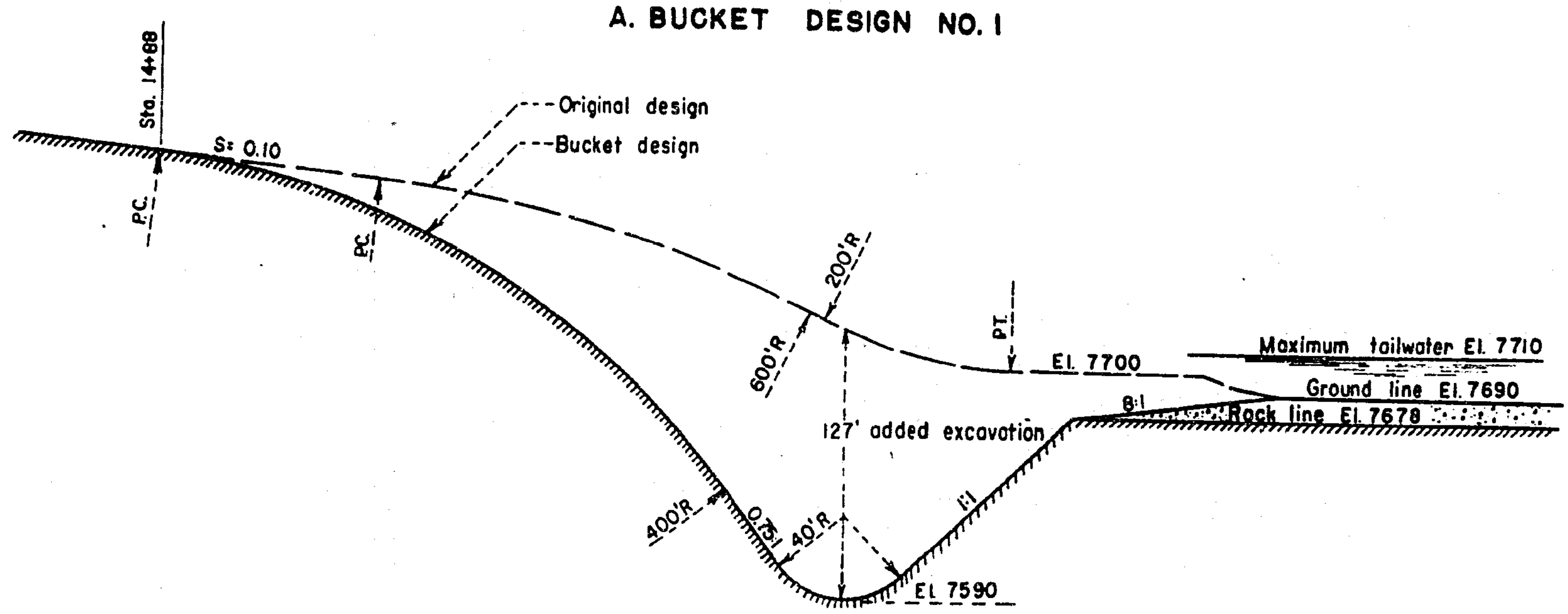
B. SCOUR FOR 7,500 SECOND-FEET.



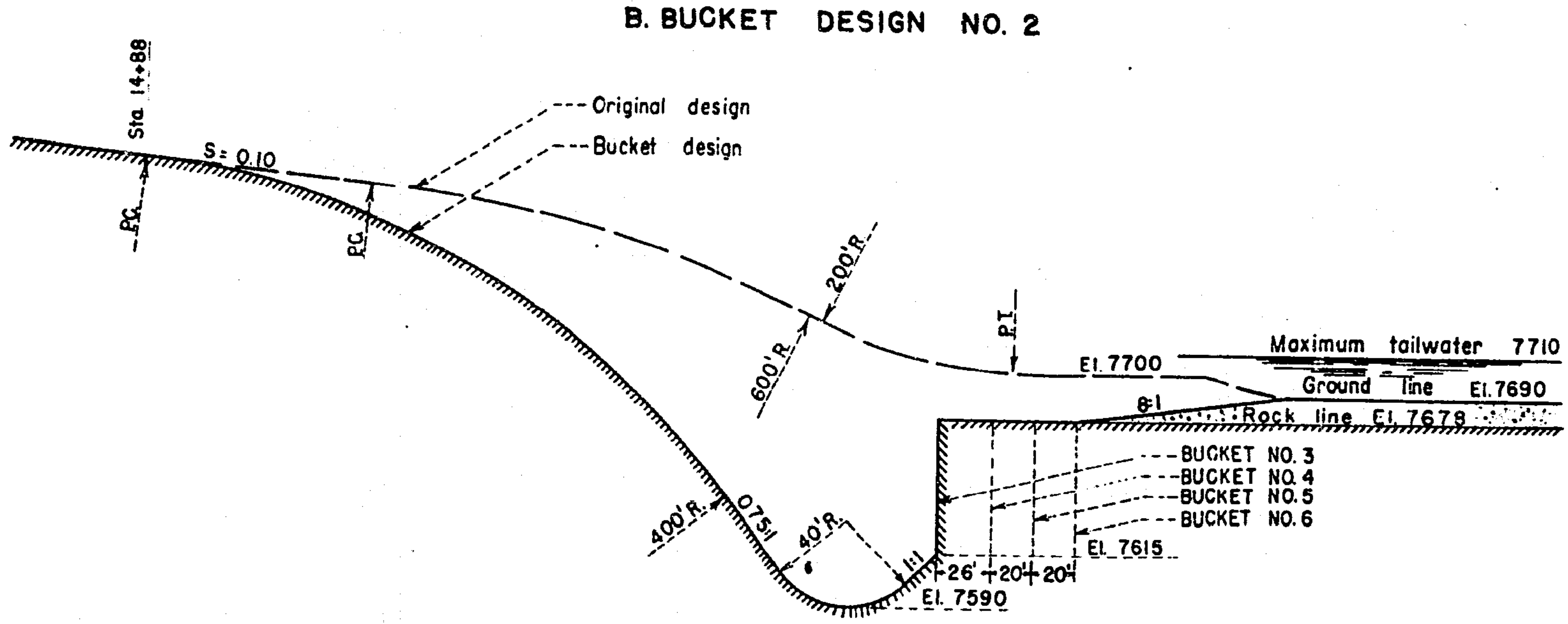
A. SCOUR FOR 5,000 SECOND-FEET



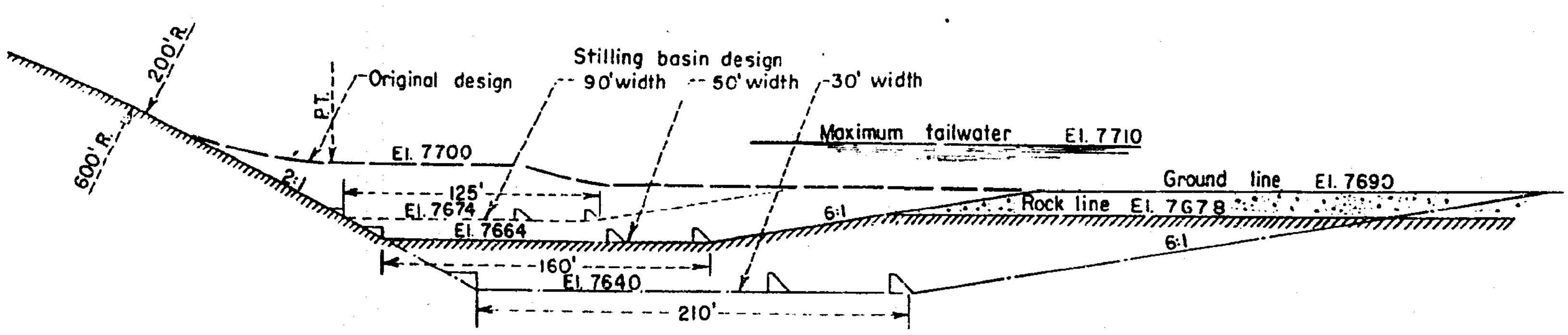
A. BUCKET DESIGN NO. 1



B. BUCKET DESIGN NO. 2

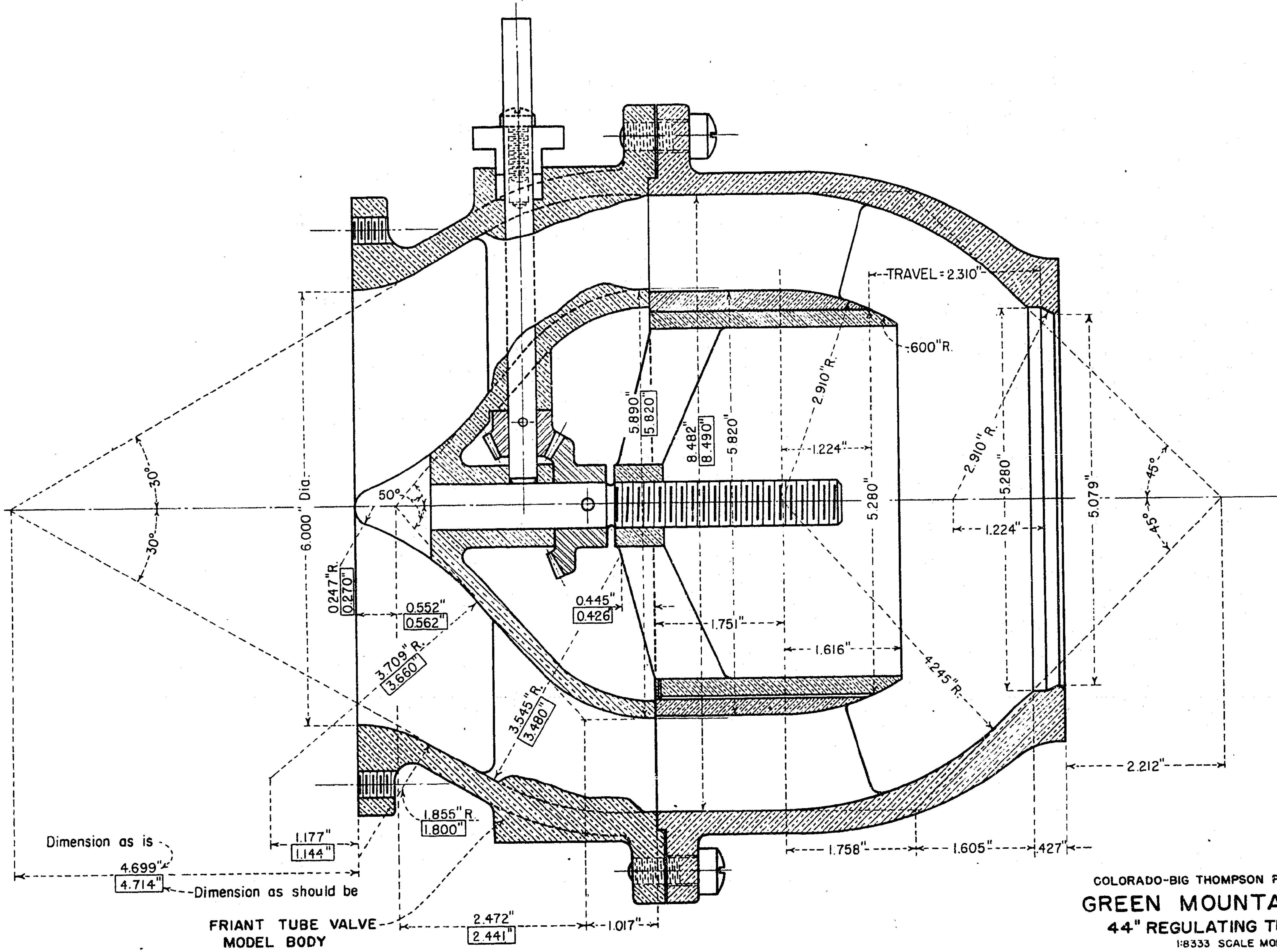


C. BUCKET DESIGN NOS. 3-6



D. STILLING BASIN DESIGN

COLORADO BIG THOMPSON PROJECT-COLO.
GREEN MOUNTAIN DAM SPILLWAY
STILLING BASIN STUDIES

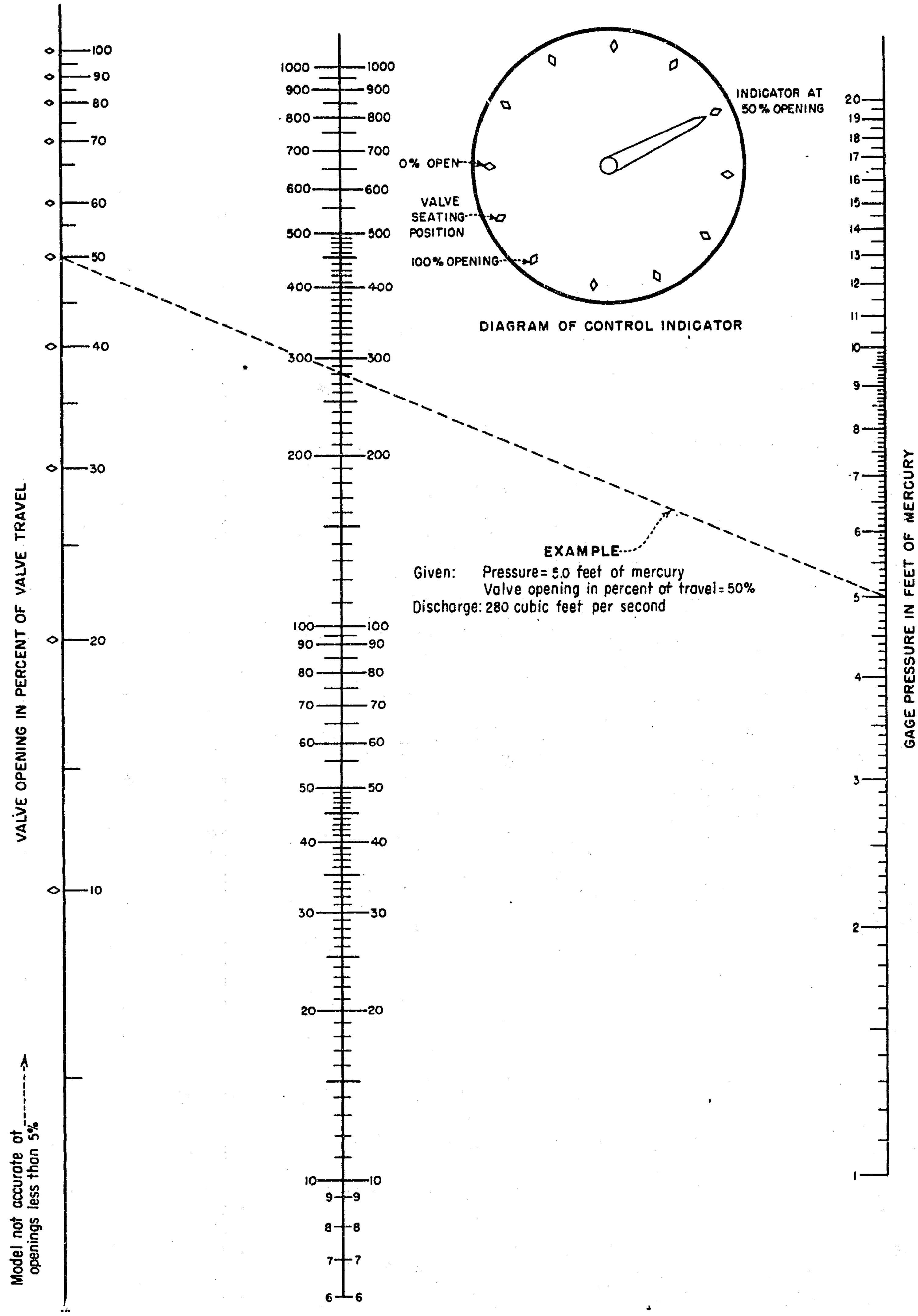


COLORADO-BIG THOMPSON PROJECT-COLO.

GREEN MOUNTAIN DAM

44" REGULATING TUBE VALVE

1:8333 SCALE MODEL



NOTE

From calibration by 6" model

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO-BIG THOMPSON PROJECT - COLO.

GREEN MOUNTAIN DAM
44" REGULATING TUBE VALVE
ALIGNMENT CHART
FOR DETERMINING DISCHARGE

DRAWN. F.C.L. ..... M.H.R. ..... SUBMITTED ....
TRACED. S.C.S. – E.W. ..... RECOMMENDED ....
CHECKED .... APPROVED ....
DENVER, COLORADO

245-D-2284

REV. 11-14-'46

FIGURE 24